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This report includes eleven papers on topics related to the development and application of computer-based mathematical models for water quality and quantity management presented at the semi-annual meeting of the Joint U.S. Canadian Stormwater Management Model Users Group held 19-20 June 1980 in Toronto, Ontario, Canada.

Topics covered include descriptions of three urban runoff models; a discussion of use of the Soil Conservation Service TR-55 model; applications of several models in planning, analysis and design; and a discussion of kinematic design storms.

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19-20 June 1980

Project Officer

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ENVIRONMENTAL RESEARCH LABORATORY
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FOREWORD

A major function of the research and development programs of the U.S. Environmental Protection Agency is to effectively and expeditiously transfer technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. The Stormwater Management Model (SWMM) users group, sponsored jointly by EPA and Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum.

This report, a compendium of papers presented at the last users group meeting, is published in the interest of disseminating to a wide audience the work of group members.

David W. Duttweiler
Director
Environmental Research Laboratory
Athens, Georgia

ABSTRACT

This report includes eleven papers on topics related to the development and application of computer-based mathematical models for water quantity and quality management presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater Management Model (SWMM) Users Group held 19-20 June 1980 in Tronto, Ontario, Canada.

Topics covered include descriptions of three urban runoff models; a discussion of the use of the Soil Conservation Service TR-55 model; applications of several models in planning, analysis and design; and a discussion of kinematic design storms.

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COMBINED SEWER SYSTEM ANALYSIS
USING STORM AND SWMM
FOR
THE CITY OF CORNWALL

by

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INTRODUCTION

The City of Cornwall, located on the shore of the St. Lawrence River, approximately 75 miles upstream of the City of Montreal, comprises a population of approximately 46,000 people and as such, is one of the major urban centres along the river throughout its length from Lake Ontario to the Atlantic Ocean.

The developed area of the City comprises a total area of approximately 4250 acres. Of this, some 1785 acres are serviced with combined sewers.

The age of the sewer system is quite variable and some sewers date back almost 100 years. The orientation of the original sewer system was such that all sewers discharged directly, without treatment, to the St. Lawrence River. During the period of the early 1960's however, these sewers were intercepted and flows directed to a new treatment facility, providing primary treatment before discharge of the treated effluent to the river.

The layout of the sewer system and the combined and separate sewer areas are shown on Figure No. 1.

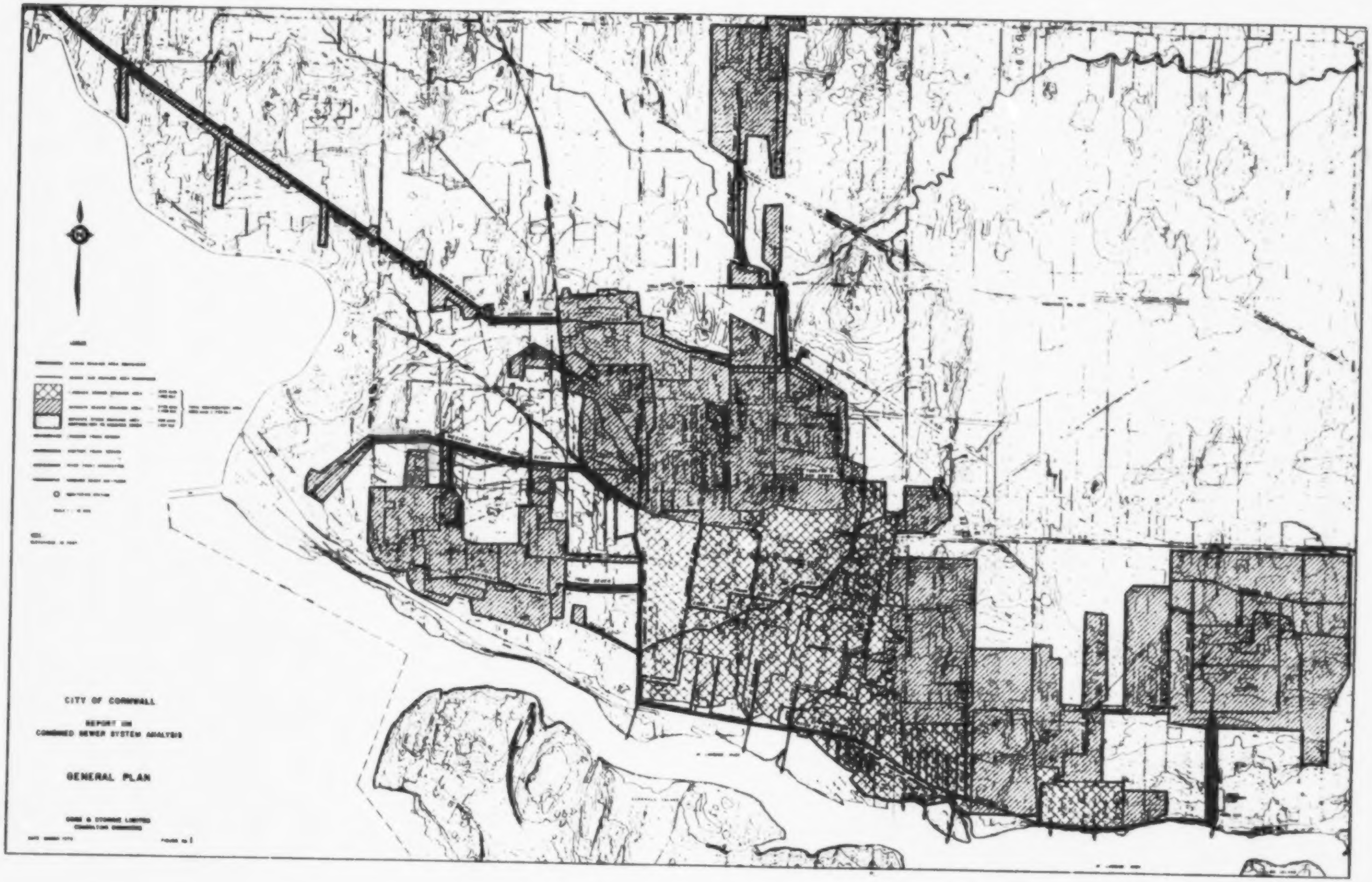
The flow from the various combined sewers are intercepted through regulator chambers, designed to intercept 2½ times dry-weather flow. All flows exceeding this amount are discharged directly through the overflow sewers to the river. A total of 8 points of overflow are provided.

PROBLEM DEFINITION

The sewage treatment facilities are designed to provide treatment for a total flow of 8.25 million imperial gallons per day. When the plant was originally commissioned, average flow rates to the plant were in the order of approximately 7 million imperial gallons per day. However, these flows increased fairly rapidly over the next few years along with the growth in the service area, to a present day average daily flow rate of approximately 11 million imperial gallons per day and a dry-weather flow rate of approximately 9 million imperial gallons per day. As such, the plant is presently operating in an overloaded condition.

Although the assimilative capacity of the St. Lawrence River as a receiving body is relatively large, high bacteria levels have been measured at various swimming beaches along the river shore, downstream from the City.

In view of these problems, it was apparent that some plant expansion was required. Beacuse of the nature of the system, however, the problem became:


CITY OF CORNWALL
REPORT ON
COMBINED SEWER SYSTEM ANALYSIS
GENERAL PLAN

- What size of plant expansion would best suit both present day and some future projected need
- What additional control techniques could be implemented to manage the system and minimize the impact of combined sewer overflow loadings to the receiving stream.

SYSTEM ANALYSIS

The approach to the analysis was to use model simulation techniques to assist in analysis and in this respect, the Storage Treatment Overflow Model "STORM" was employed as the primary tool.

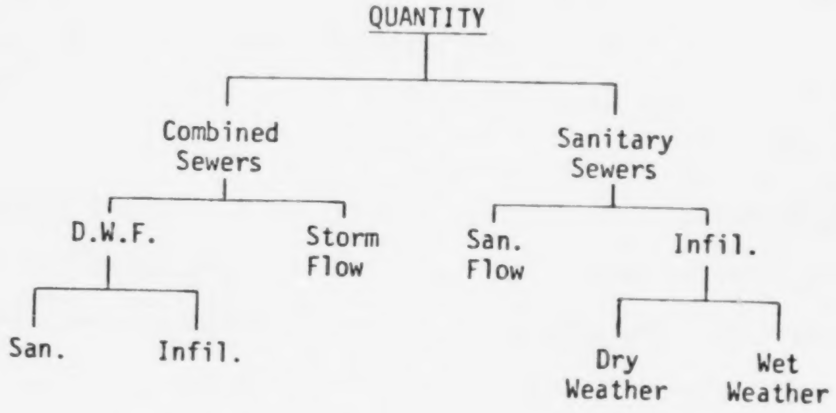
This model was used to:

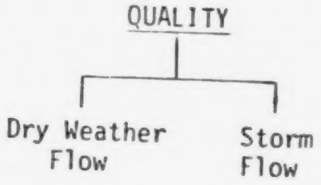
- Screen a number of control alternatives
- Develop statistics with respect to system operation for various levels of both storage and treatment capacities.

Subsequently, the Storm Water Management Model "SWMM" was used to assess selected control alternatives under individual event operation.

The first step in the application of the models was to undertake some calibration and subsequent testing of the calibration, both with respect to quantity and quality of flow. For this purpose, a number of typical sub-catchment watershed areas were identified both large and small, and data was collected over the period of approximately 1 summer season. In addition, both flow and quality data were available from the water pollution control plant records for assessment of both dry weather and average daily flow conditions.

Quantity and Quality data required is summarized as follows:


QUANTITY
Combined Sewers
Sanitary Sewers
D.W.F.
Storm Flow
San. Flow
Infil.
San.
Infil.
Dry Weather
Wet Weather


QUALITY
Dry Weather Flow
Storm Flow

For the sanitary and infiltration components of dry-weather flow, rates were calculated from the measured hydrographs, using essentially the minimum night time flow condition, when the majority of the flow is attributed, particularly in the smaller areas, to infiltration. Average sanitary components were also compared to water consumption and favourable correlation was obtained.

Specific major industrial sources were identified separately, along with associated flow rates and sewage strengths.

Table No. 1 following is a comparison of both simulated and measured flow and quality data, considering:

- Dry Weather Flow
- Average Daily Flow
- Annual Overflow Volume
- Sewage Strengths

Input to the Storm Model included both quantity and quality information relating to dry-weather flow for domestic and industrial purposes, as well as infiltration and extraneous flows. The model then subsequently produced predictions of average daily flow rates, along with associated quality parameters and annual overflow volumes.

It is noted on examination of Table No. 1, that both the predicted average daily flow volumes and associated pollutant concentrations are within a reasonable range of measured values. It is also noted, however, that the volume of overflow predicted is significantly lower than the volume of overflow actually measured.

Further examination revealed this to be attributed to two problems:
The first problem related to the predicted volumes of overflow for spring runoff conditions. The calibration of the Storm Model was a relatively straightforward process for summertime occurrences and for this purpose, calibration was undertaken by adjusting the average runoff coefficients for both pervious and impervious areas, until a reasonable fit was obtained. Subsequent testing of the calibration on additional storm events indicated good and reasonable correlation. The results of testing on 2 separate storm events are shown on Figure Nos. 2 and 3.

Because the operation of the system was investigated on an average annual basis, as well as seasonal variations, and in view of the geographic location of the

City of Cornwall, it was necessary to utilize also, the snowmelt computation capabilities of the model. However, because of frozen ground conditions during the spring runoff season, it was necessary to consider increased runoff coefficients during this period of time, over and above those values applied to

TABLE NO. 1
CITY OF CORNWALL
WATER POLLUTION CONTROL PLANT
Measured vs. Modelled Values

	Avg. Year Precipitation (in.)	DWF Flow (mgd)	Average Daily Flow (mgd)	Brookdale Weir Overflow (mg/year)	Influent Pollutant Conc. (mg/L)		
					S.S.	B.O.D.	P
Recorded (WPCP)							
1969	-	-	6.9	-	140	52	-
1970	-	-	7.4	-	120	80	-
1971 (avg.)	-	-	8.4	-	155	80	5.8
1972 (avg.)	-	-	9.8	-	185	110	5.2
1973 (avg.)	46.3	-	11.0	165	160	140	3.2
1974 (avg.)	36.1	8.7	10.4	128	160	141	3.6
1975 (avg.)	34.7	8.3	10.5	73	171	177	3.7
1976 (avg.)	46.0	9.1	11.1	178	186	125	3.6
1977 (avg.)	47.3	9.7	12.2	358	221	127	4.6
1978 (avg.)	36.4	8.2	11.1	560	121	134	4.4
1979 (avg.)	37.8	8.5	10.7	298	110	106	3.7
Average Values (1974 - 1979)		8.8	11.0	265	161	135	3.9
STORM MODEL INPUT DWF ONLY*							
Res./Comm - Domestic		2.6	-	-	376	283	11.0
Ind.		.9	-	-	1130	763	17.4
Infiltr. + Extraneous		5.4	-	-	-	-	-
Average		8.9			224	160	4.9
STORM MODEL OUTPUT for Avg. Precip. Year**		-	10.9	68.5	197	130	4.3

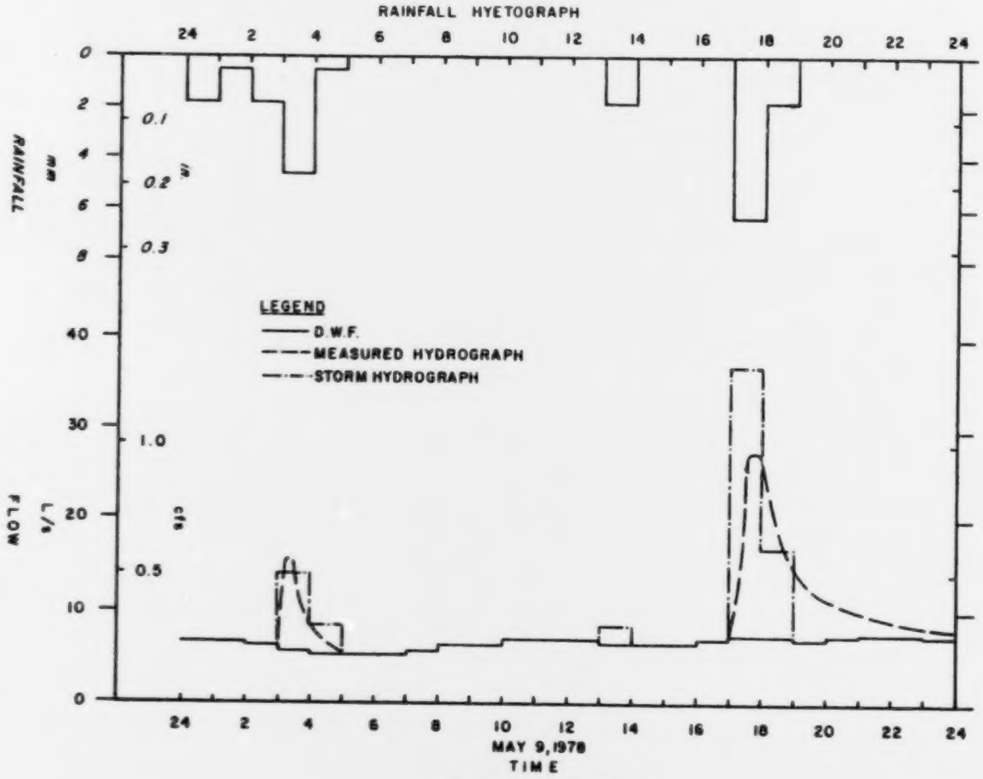
* Refers to Res., Comm., Indust,. & Infiltration Flows Combined during dry weather periods indicating average daily dry weather flows and average quality concentrations.

** Indicates the average quality resulting from the DWF combined with runoff due to precipitation for an average of 10 years precipitation records.

the summer season. This was particularly important for pervious areas of the watershed where it was found necessary to increase the runoff coefficient from .04 to .44 - a factor of 10.

This experience, along with other projects of a similar nature, indicate the need for a variable coefficient and/or soil complex curve number input into the model.

FIGURE NO. 2



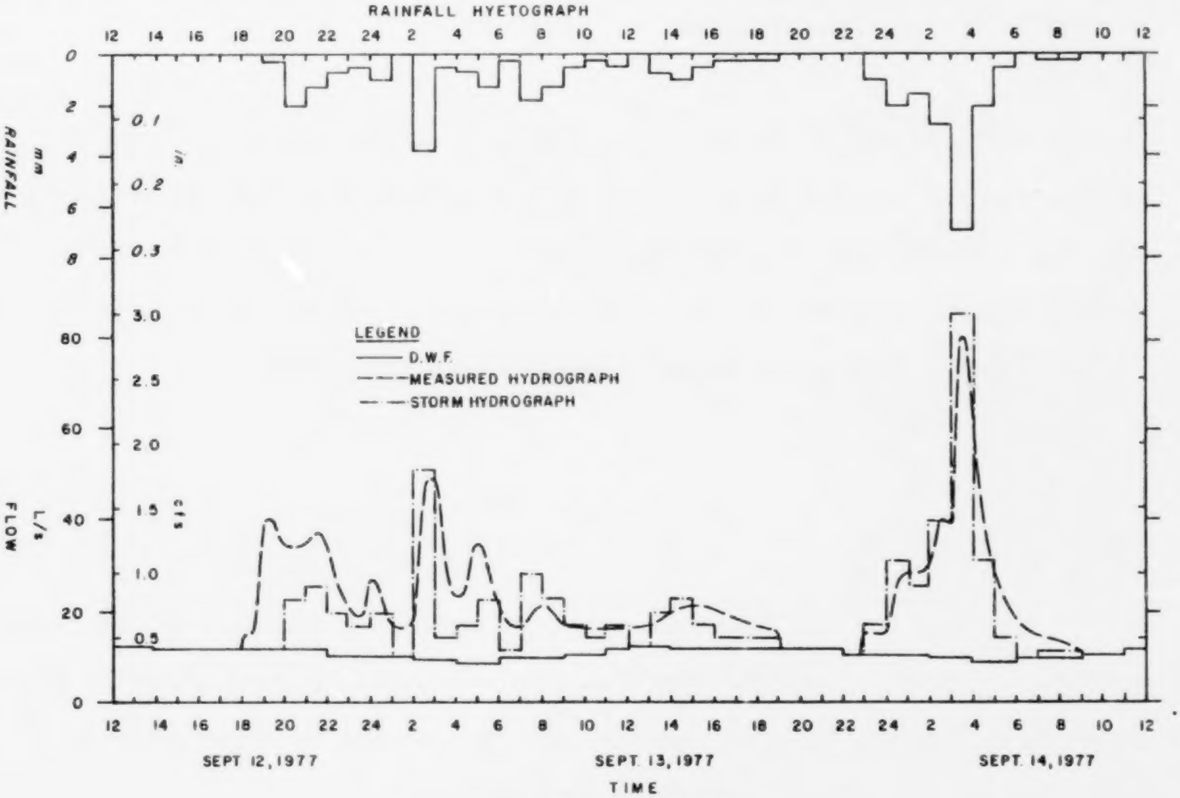
CITY OF CORNWALL

REPORT ON COMBINED SEWER SYSTEM ANALYSIS

TESTING OF CALIBRATED STORM
AMELIA ST. COMBINED SEWER CATCHMENT

In addition, it was found necessary also, to decrease the melt coefficient somewhat, measured as inches per day per degree Fahrenheit from .07 to .055. This modification was necessary as the simulated melt was occurring too early and decreasing the melt coefficient both delayed the peak and increased the total duration of the melting time.

FIGURE NO. 3



CITY OF CORNWALL

REPORT ON COMBINED SEWER SYSTEM ANALYSIS

TESTING OF CALIBRATED STORM
AUGUSTUS ST. COMBINED SEWER CATCHMENT

The second problem related to the calibration of the overflow weir where combined sewer overflow events are measured. It was found that the calibration of the overflow weir required significant modification as readings were continuously high.

After the modifications as outlined above, a simulation of the system operation was undertaken over the period of 1 year, comparing both volume of overflow and number of events to those actually measured after recalculating overflows on the basis of the new weir calibration. The results of this comparison are shown in Table No. 2 and it is noted that reasonable correlation, both in terms of volume and number of events was obtained.

The simulated volume was somewhat less than that actually measured by about 6%, whereas the total number of events was in fact greater than that measured by some 27%. It is noted, however, that the number of events simulated included for various months, a number of minor overflow events, which were not actually recorded but rather were taken up by natural storage within the system.

CONTROL ALTERNATIVES

A listing of various control alternatives was produced and these included various levels of treatment plant capacity expansion, in-system storage, diversion of storm flows from the combined sewer system, improvements in street sweeping practices and industrial flow control or abatement.

A preliminary screening was undertaken of these various alternatives on the basis of present day dry-weather flow rates. The results of this screening are summarized in Table No. 3. The screening was undertaken by comparing the effectiveness of each alternative in comparison with the present day system,

TABLE NO. 2

CITY OF CORNWALL

COMBINED SEWER ANALYSIS

1 Year Comparison - Actual vs. Simulated
(May 1978 - April 1979)

Month	Measured - Brookdale Overflow		S.T.O.R.M. Predictions	
	Volume (M.G.)	No. of Events	Volume (M.G.)	No. of Events
May	4.423	6	2.73	5
June	.929	4	1.51	6
July	.714	3	1.06	3
August	7.309	5	8.49	7*
September	1.699	2	2.42	7*
October	4.309	3	5.00	3
November	1.94	2	3.64	2
December	-	-	-	-
January	10.641	2	8.95	5*
February	.207	1	4.25	2
March	54.174	14	43.32	13
April (20th)	12.943	5	12.28	7
Total	99.288	47	93.65	60

* includes a number of minor overflow events simulated but not actually measured

TABLE NO. 3

COMPARISON LISTING OF CONTROL ALTERNATIVES

	STORM WATER MANAGEMENT CONTROL SCHEME	TOTAL ANNUAL LOADING (FROM WATERSHED)				OVERFLOW - ANNUAL LOADING					FLOW TO W.P.C.P.				WPCP-EFFLUENT DISCHARGE				NET LOAD TO RIVER				% REDUCTION FROM PRESENT SYSTEM		
		FLOW $FT^3 x10^6$	S S $LBS x10^6$	BOD $LBS x10^6$	P $LBS x10^6$	FLOW $FT^3 x10^6$	FLOW NO OF EVENTS	S S $LBS x10^3$	BOD $LBS x10^3$	P $LBS x10^3$	FLOW $FT^3 x10^6$	S S $LBS x10^6$	BOD $LBS x10^6$	P $LBS x10^6$	FLOW $FT^3 x10^6$	S S $LBS x10^6$	BOD $LBS x10^6$	P $LBS x10^6$	FLOW $FT^3 x10^6$	S S $LBS x10^6$	BOD $LBS x10^6$	P $LBS x10^6$	S S	BOD	P
1	PRESENT SYSTEM - 8.25 MGD TREATMENT 24.0 MGD HYDRAULIC	651.6	7.611	5.262	0.175	27.21	50	171	305.4	5.61	624.4	7.440	4.957	0.169	624.4	(25%) 5.580	(10%) 4.461	(20%) .135	651.6	5.751	4.766	.1406	-	-	-
2	PLANT EXPANSION - 24.0 MGD TREATMENT AND HYDRAULIC	651.6	7.611	5.262	0.175	27.21	50	171	305.4	5.61	624.4	7.440	4.957	0.169	624.4	(80%) 1.488	(30%) 3.470	(80%) .034	651.6	1.659	3.775	.0396	71	21	72
3	PLANT EXPANSION - 36 MGD	651.6	7.611	5.262	0.175	16.61	33	111.6	179.3	3.32	635.0	7.499	5.083	0.172	635.0	(80%) 1.500	(30%) 3.558	(80%) .034	651.6	1.612	3.379	.0372	72	29	74
4	USE OF EXISTING TRUNK SEWER STORAGE *	651.6	7.611	5.262	0.175	16.68	19	87.7	115.2	2.30	634.9	7.523	5.147	0.173	634.9	(25%) 5.642	(10%) 4.632	(20%) .138	651.6	5.730	4.747	.1400	0.4	0.4	0.4
5	DIVERT 433 AC STORM TO FLY CREEK	564.9	6.756	4.606	0.158	24.71	46	133.8	240.9	4.41	540.2	6.622	4.365	0.154	540.2	(25%) 4.967	(10%) 3.929	(20%) .123	564.9	5.100	4.169	.1273	11	13	9
6	IMPROVED STREET SWEEPING	651.6	7.495	4.906	0.168	27.21	49	110.1	191.0	3.72	624.4	7.385	4.715	0.164	624.4	(25%) 5.539	(10%) 4.244	(20%) .131	651.6	5.649	4.435	.1351	2	7	4
7	INDUSTRIAL FLOW ELIMINATED	595.8	3.967	2.515	0.104	26.20	49	139.3	272.2	4.91	569.6	3.828	2.243	0.099	569.6	(25%) 2.871	(10%) 2.019	(20%) .079	595.8	3.101	2.291	.0842	48	52	40
8	INDUSTRIAL FLOW @ BYLAW LIMITS	651.6	5.190	3.551	0.175	27.21	49	151.6	288.5	5.61	624.4	5.038	3.263	0.169	624.4	(25%) 3.779	(10%) 2.936	(20%) .135	651.6	3.930	3.225	.1406	32	32	-
9	PLANT EXPANSION - 24.0 MGD + STORAGE IN EXISTING TRUNK *	651.6	7.611	5.262	0.175	16.68	19	87.7	115.2	2.30	634.9	7.523	5.147	0.173	634.9	(80%) 1.505	(30%) 3.603	(80%) .035	651.6	1.592	3.718	.0369	72	22	74
10	PLANT EXPANSION 24.0 MGD + STORAGE IN EXISTING TRUNK + DIVERT 433 AC TO FLY CREEK *	564.9	6.756	4.606	0.158	13.17	17	61.6	78.6	1.57	551.7	6.694	4.527	0.156	551.7	(80%) 1.339	(30%) 3.169	(80%) .031	564.9	1.400	3.248	.0329	76	32	77
11	PLANT EXPANSION - 24.0 MGD + STORAGE IN EXISTING TRUNK + IMPROVED STREET SWEEPING *	651.6	7.495	4.906	0.168	16.68	19	52.6	74.8	1.58	634.9	7.442	4.831	0.166	634.9	(80%) 1.488	(30%) 3.382	(80%) .033	651.6	1.541	3.434	.0349	73	28	75
12	PLANT EXPANSION - 24.0 MGD + STORAGE IN EXISTING TRUNK + IMPROVED STREET SWEEPING + INDUSTRIAL FLOW @ BYLAW LIMITS	651.6	5.102	3.198	0.168	16.68	19	42.5	67.6	1.58	634.9	5.060	3.120	0.166	634.9	(80%) 1.012	(30%) 2.191	(80%) .033	651.6	1.054	2.259	.0349	82	53	75
13	PLANT EXPANSION - 36 MGD + STORAGE IN EXISTING TRUNK *	651.6	7.611	5.262	0.175	9.05	15	52.8	62.8	1.26	642.5	7.558	5.199	0.174	642.5	(80%) 1.512	(30%) 3.639	(80%) .035	651.6	1.564	3.702	.0360	73	22	74
14	PLANT EXPANSION 36 MGD + STORAGE IN EXISTING TRUNK + DIVERT 433AC TO FLY CREEK *	564.9	6.756	4.606	0.158	5.47	13	31.3	34.2	0.68	559.4	6.725	4.572	0.157	559.4	(80%) 1.345	(30%) 3.200	(80%) .031	564.9	1.376	3.234	.0321	76	32	77
15	PLANT EXPANSION - 36 MGD + STORAGE IN EXISTING TRUNK + IMPROVED STREET SWEEPING *	651.6	7.495	4.906	0.168	9.05	15	29.2	39.5	0.83	642.5	7.466	4.867	0.167	642.5	(80%) 1.493	(30%) 3.407	(80%) .033	651.6	1.522	3.446	.0343	74	28	76
16	PLANT EXPANSION - 36 MGD + STORAGE IN EXISTING TRUNK + IMPROVED STREET SWEEPING + INDUSTRIAL FLOW @ BYLAW LIMITS *	651.6	5.102	3.198	0.168	9.05	15	11.4	35.9	1.58	642.5	5.091	3.162	0.166	642.5	(80%) 1.018	(30%) 2.213	(80%) .033	651.6	1.030	2.249	.0343	82	53	76

* STORAGE IN EXISTING SEWER = 355,000 FT^3 TO ELEV 187.4

NOTE (_%) SIGNIFIES PLANT EFFICIENCY

considering as a yardstick, the reduction in total load discharged to the receiving stream, considering both treatment plant effluent discharge and combined sewer. Examination of this preliminary evaluation indicates that the most promising combination of alternatives is some combination of storage and treatment.

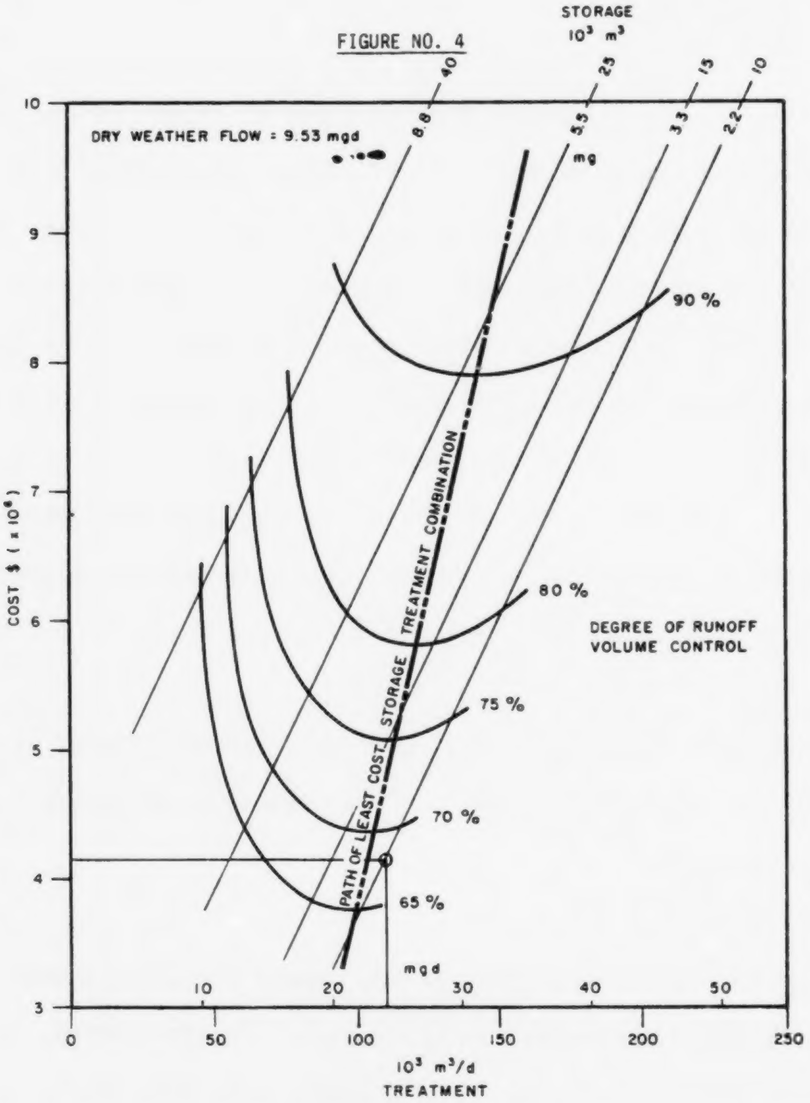
The Storm Model was then utilized further in order to assess various levels of storage and treatment, first on the basis of present dry weather flow conditions (i.e. 9.5 MIGD) and then secondly, on a future increased dry weather flow. The lowest level of storage considered was that level of storage naturally available within the Riverfront Interceptor, above the natural depth of flow for dry weather flow conditions. A family of curves was then produced, showing the relationship of varying levels of treatment rates in conjunction with storage volumes, both as a function of capital cost and as a function of degree of control, measured in terms of runoff from the watershed directed through storage and treatment. It is important also to note that the cost functions are based on the premise that:

a) The available storage in the Riverfront Interceptor sewer in the amount of 2.2 million gallons, is available and can be provided at minimum cost.

b) The cost of treatment expansion only, above the present capacity of 8.25 MIGD is included in the analysis. In other words, it is assumed that the existing facility has already been amortized.

Figure No. 4 shows that an increase in treatment capacity to a 24 MIGD rate in conjunction with utilization of the available trunk interceptor capacity will provide a degree of treatment for approximately 67% of the stormwater runoff. As the dry weather flow increases, this level of control decreases to approximately 62%, as noted on Figure No. 5.

FIGURE NO. 4



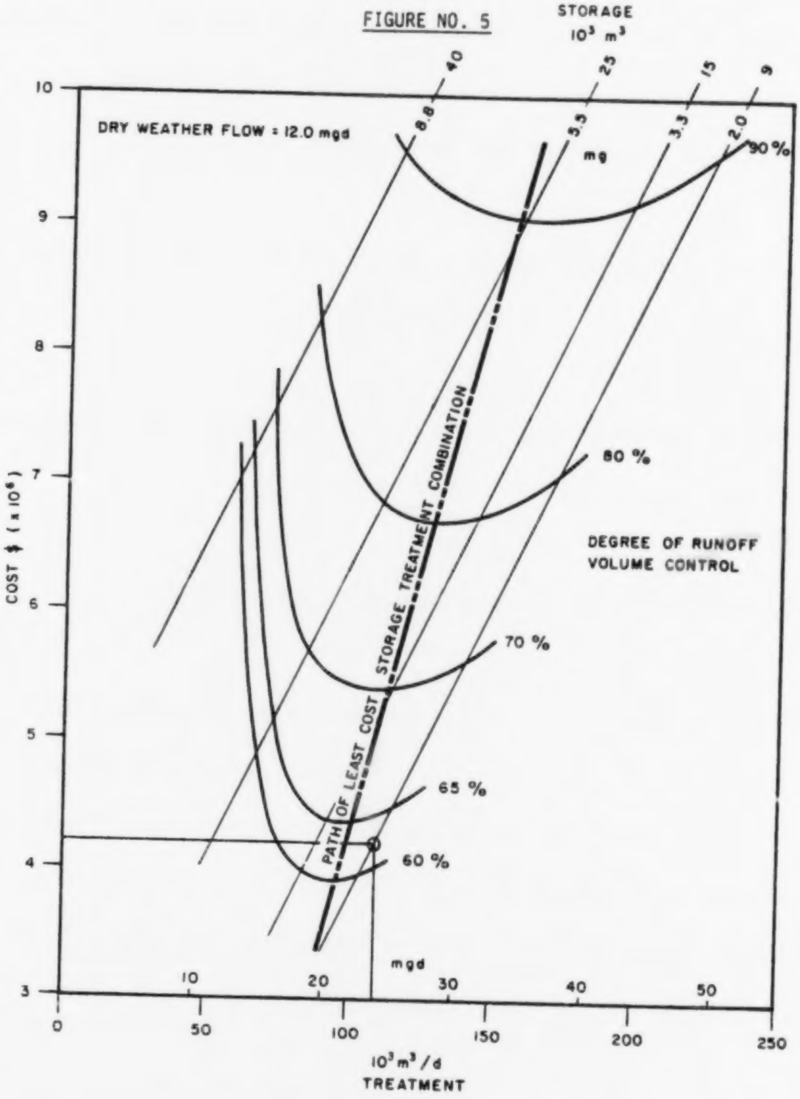
CITY OF CORNWALL

REPORT ON COMBINED SEWER SYSTEM ANALYSIS

STORAGE TREATMENT COMBINATION
PRESENT FLOW EFFECTIVENESS

It is noted on both of these figures, that although the selected combinations do not fall on the theoretical "least cost" line, the 24 MIGD treatment rate is a function of the present plant size in terms of settling tank units and as such, is a realistic size of expansion to consider. At the same time, it can readily

FIGURE NO. 5



CITY OF CORNWALL
REPORT ON COMBINED SEWER SYSTEM ANALYSIS
STORAGE TREATMENT COMBINATION
FUTURE FLOW EFFECTIVENESS

be seen that if desired, higher levels of control can be most economically provided by additional storage in the system.

It is important to consider also the trend in the "level of control", or the number and volume of combined sewer overflow events as development takes place within the City and normal dry weather flow rates increase. For comparison purposes, a future condition dry weather flow of 12.0 MIGD was assessed, which represents an increase in present day development of about 25%.

Considering the downstream beaches as a focal point of concern, the seasonal impact of these overflows is also important.

Figure No. 6 illustrates the projected trend in both total volume and number of events of combined sewage overflows considering:

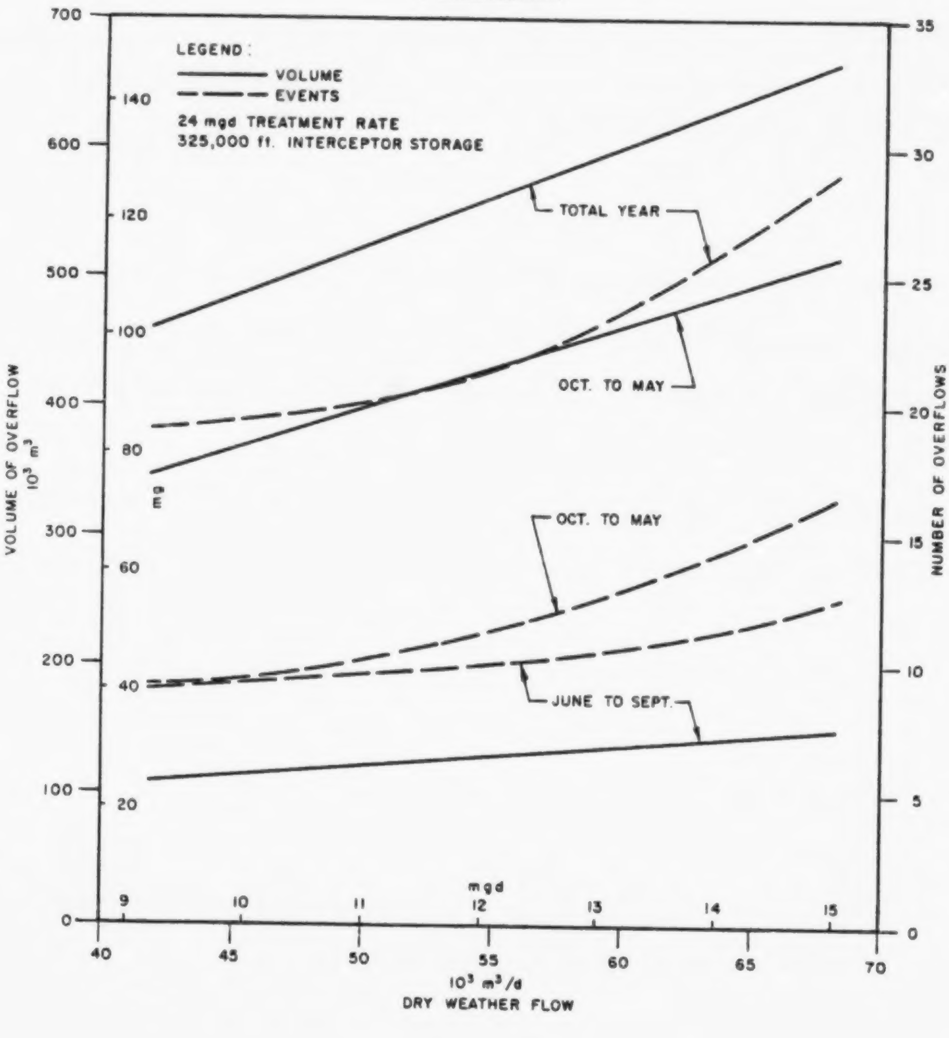
- A 24 MIGD treatment rate
- Existing available interceptor sewer storage.

It is noted that, during the critical summer season, the overflow occurrences will average approximately 9 - 10 under present day conditions and this will increase to 11 or 12 overflows as dry weather flows increase to 12 MIGD in the future. The associated volumes of overflow will also increase from about 25 million gallons under present day flow conditions to about 30 million gallons for the future 12 MIGD dry weather flow condition.

For comparison purposes, Table No. 4 shows the control effectiveness of a 24 MIGD treatment capacity in combination with storage, as opposed to a 36 MIGD treatment rate without storage.

It is noted that the lower treatment rate in conjunction with storage provides a higher level of control with respect to total number of events but a slightly

FIGURE NO. 6



CITY OF CORNWALL

REPORT ON COMBINED SEWER SYSTEM ANALYSIS

PROPOSED STORAGE TREATMENT COMBINATION
VOLUME / NUMBER OVERFLOWS FOR INCREASING DRY WEATHER FLOWS

TABLE NO. 4

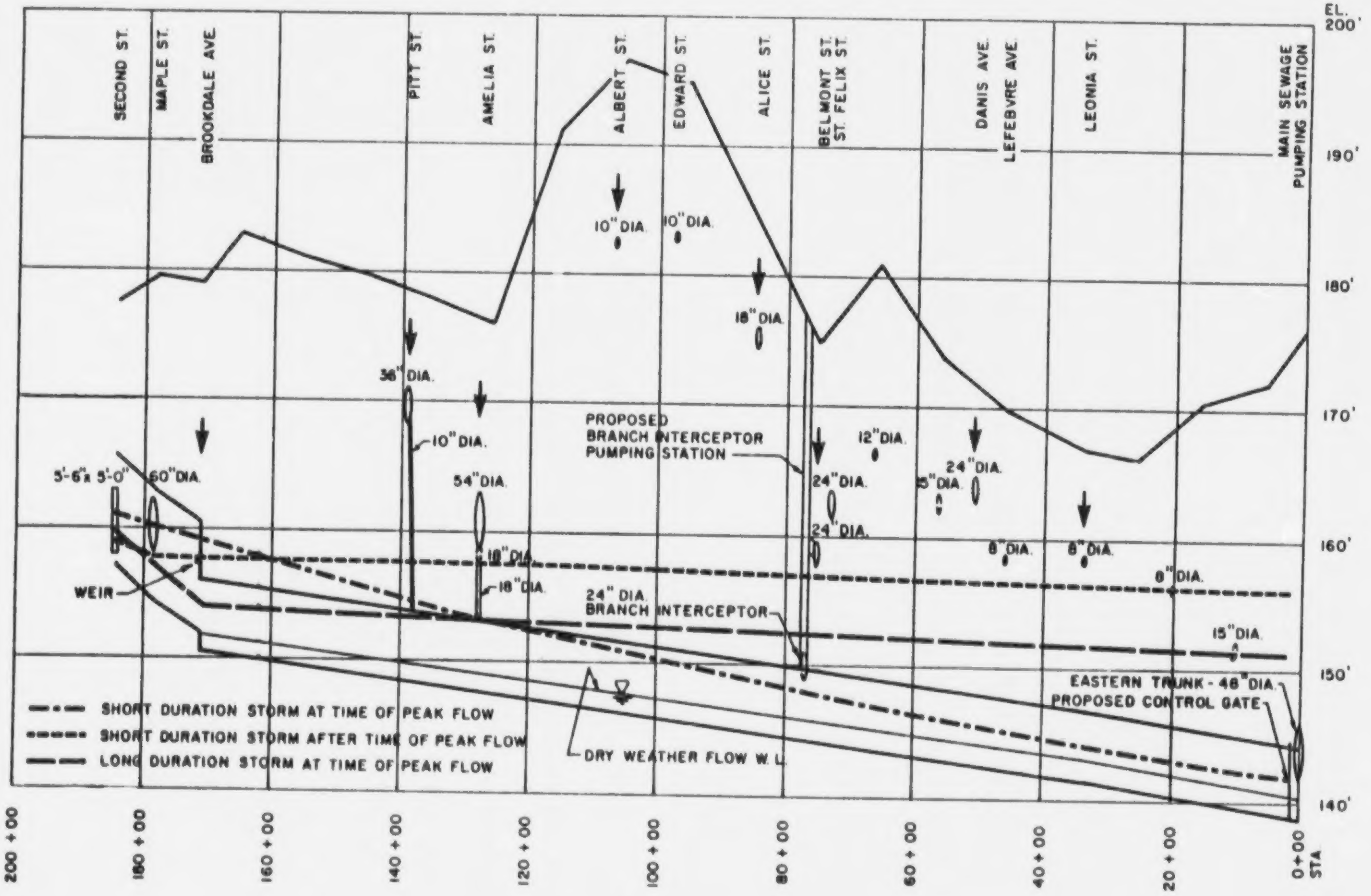
	24 MIGD Treatment + Trunk Sewer Storage	36 MIGD Treatment
OVERFLOW EVENTS		
a) Summertime (June-September)	10	16
b) Remainder of year (October-May)	12	20
c) Total	22	36
OVERFLOW VOLUMES		
a) Summertime (June-September)	25 MG	35 MG
b) Remainder of year (October-May)	95 MG	75 MG
c) Total	120 MG	110 MG

lower level of control considering total volume. This is accounted for by the fact that the treatment-storage combination totally contains more of the smaller volume events which create overflow for the 36 MIGD treatment rate alone.

The storage treatment combination does not provide as effective a control during periods of spring runoff and hence, the volume of overflow is somewhat higher for this control scheme.

PRINCIPLE OF OPERATION

Figure No. 7 is a profile of the interceptor sewer showing the various points of interception. As a principle of operation, to effect maximum control, it is most desirable that the points of overflow be centralized to perhaps one or two locations. In the event then, that higher levels of control are deemed necessary, additional storage can more readily and effectively be provided and chlorination and/or diffusion facilities may also be readily installed.


SECOND ST.
MAPLE ST.
BROOKDALE AVE.
PITT ST.
AMELIA ST.
ALBERT ST.
EDWARD ST.
ALICE ST.
BELMONT ST.
ST. FELIX ST.
DANIS AVE.
LEFEBVRE AVE.
LEONIA ST.
MAIN SEWAGE PUMPING STATION
EL. 200'
190'
180'
170'
160'
150'
140'
10" DIA.
10" DIA.
18" DIA.
36" DIA.
10" DIA.
54" DIA.
18" DIA.
18" DIA.
PROPOSED BRANCH INTERCEPTOR PUMPING STATION
12" DIA.
24" DIA.
24" DIA.
15" DIA.
24" DIA.
8" DIA.
8" DIA.
8" DIA.
15" DIA.
5'-6" x 5'-0"
60" DIA.
WEIR
24" DIA. BRANCH INTERCEPTOR
EASTERN TRUNK - 48" DIA.
PROPOSED CONTROL GATE
DRY WEATHER FLOW W. L.
SHORT DURATION STORM AT TIME OF PEAK FLOW
SHORT DURATION STORM AFTER TIME OF PEAK FLOW
LONG DURATION STORM AT TIME OF PEAK FLOW
200 + 00
180 + 00
160 + 00
140 + 00
120 + 00
100 + 00
80 + 00
60 + 00
40 + 00
20 + 00
0 + 00
STA

CITY OF CORNWALL
PROFILE OF RIVER FRONT INTERCEPTOR

SCALE : HOR. : 1" = 2000'
VERT. : 1" = 10'

FIGURE NO. 7

The most appropriate location for such centralized facilities would appear to be at the upstream end of the interceptor sewer at Brookdale Avenue at which point, approximately 72% of the combined sewer area is tributary to the interceptor.

Considering tributary areas, flows, sewer capacities and storage volumes, an initial conclusion was that all five easterly overflows could be closed and the entire wet weather flow intercepted. The tributary areas and flows at Pitt Street and particularly Amelia Street, however, indicated that more analysis of the hydraulic consequences of closing these two overflows was warranted.

Further analysis was undertaken using the extended transport block of the Storm Water Management Model - SWMM - in order to assess the hydraulic operation of the system under individual event operation.

As a starting point, two historical storms were chosen with total rainfall volumes in the order of 1.5 inches, which is about equivalent to a 2 year return event in terms of total volume. The two storms chosen were:

- 1.51 inches over a duration of 4 hours
- 1.43 inches over a duration of 9 hours.

The shorter duration storm naturally produced larger peak flow rates. As an initial step, the analysis assumed total interception at all five easterly overflow locations and three times dry weather flow rates at both Pitt Street and Amelia Street.

The hydraulic performance of the system operation for the shorter duration storm is shown on Figure No. 7. The lower water surface profile is at the time of peak flow occurrence in the Brookdale Avenue trunk and consequently, the time of peak overflow rate. It is noted at this time, the available storage has not yet been fully utilized. The higher water surface (hydraulic grade line) profile is at a later time period during the storm when peak flows have subsided. At this time storage is fully utilized. The problem with this operation is that maximum utilization of storage to contain "first flush" effects has not been achieved.

The second storm event analyzed was of equivalent volume but longer duration. Because of this, peak flow rates were also significantly lower and in fact, no overflow was created at the Brookdale Avenue chamber. The profile shows the maximum hydraulic grade line profile in the interceptor sewer for this event also and it is noted that full storage utilization was achieved for this event.

The next step in the analysis procedure will be to undertake a sensitivity analysis considering both various levels of "design storms" and various levels of available storage in order to determine:

a) Proper and realistic levels of interception for both the Pitt Street and Amelia Street overflows.

b) Frequency of overflows associated with such intercepting capacity at each of these two locations.

c) Maximum utilization of available storage.

This procedure is now underway.

In summary, the use of such modelling techniques provides for a study of this nature, a tool which allows analysis of a number of alternatives in such detail as would not otherwise be realistically possible and as such, permits selection of appropriate control schemes considering both costs and effectiveness. At the same time, analysis of the system operation under real storm event conditions allows the analyst to see how the system will react, pick out potential problem areas and make the necessary adjustments in deriving the best cost effective scheme.

CONSIDERATIONS REGARDING THE APPLICATION OF SCS TR-55 PROCEDURES FOR RUNOFF COMPUTATIONS

P. Wisner,[1] S. Gupta,[2] and A. Kassem[3]

INTRODUCTION

The U.S. Soil Conservation Service (SCS) Technical Release Number 55 "TR-55" (12) published in 1975 presents simplified tabular and graphical methods for estimating runoff volumes, peak discharges and discharge hydrographs for runoff computations. The method which has been applied mainly in the U.S. (8) has also been recommended for application in Canada (7).

The TR-55 does not provide any comparisons with real measurements or more sophisticated computations. Mention is made that the methods for computing peak discharges and hydrographs using time of concentration (T_c) and travel time (T_t) are approximations of the detailed hydrograph analysis, SCS TR-20 (Computer Program for Project Formulation Hydrology).

The implementation of SWM has led to the need for more sophisticated computations than the Rational Method (RM).

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Many "simplified techniques" were developed and are popular for various reasons. In a previous SWMM users group meeting in Montreal, we have indicated the need for standardization and testing of these methods and comparison with more sophisticated ones. In the Gainsville meeting, we assessed the RM based on comparisons with SWMM (18).

The intent of the paper is to present a critical review of the SCS TR-55 computational procedures and data requirements. A comparative assessment of peak discharge and hydrograph computations with other widely used models are presented. The comparisons are carried out using several hypothetical watersheds. Several limitations of the procedures are also discussed. Some of the results are taken from other detailed studies conducted by the writers (5,17).

A CRITICAL REVIEW OF THE SCS TR-55 RUNOFF COMPUTATIONS

Figure 1 is a flow chart of the TR-55 procedures for runoff computations. The following is a discussion of various steps and/or input requirements in the procedure.

(1) Meteorological Input:

The computation of peak discharges for urban and rural areas using either the graphical or the tabular methods in TR-55 are based on a 24-hour SCS Type II storm distribution. The distribution is not documented in the release. As shown in Figure 2, a "Chicago" type storm distribution as well as real storms may be more "peaky" than the SCS Type II Storm (16), where peakier storms may be more critical for the analysis of small watersheds. Therefore in urbanized watersheds, where

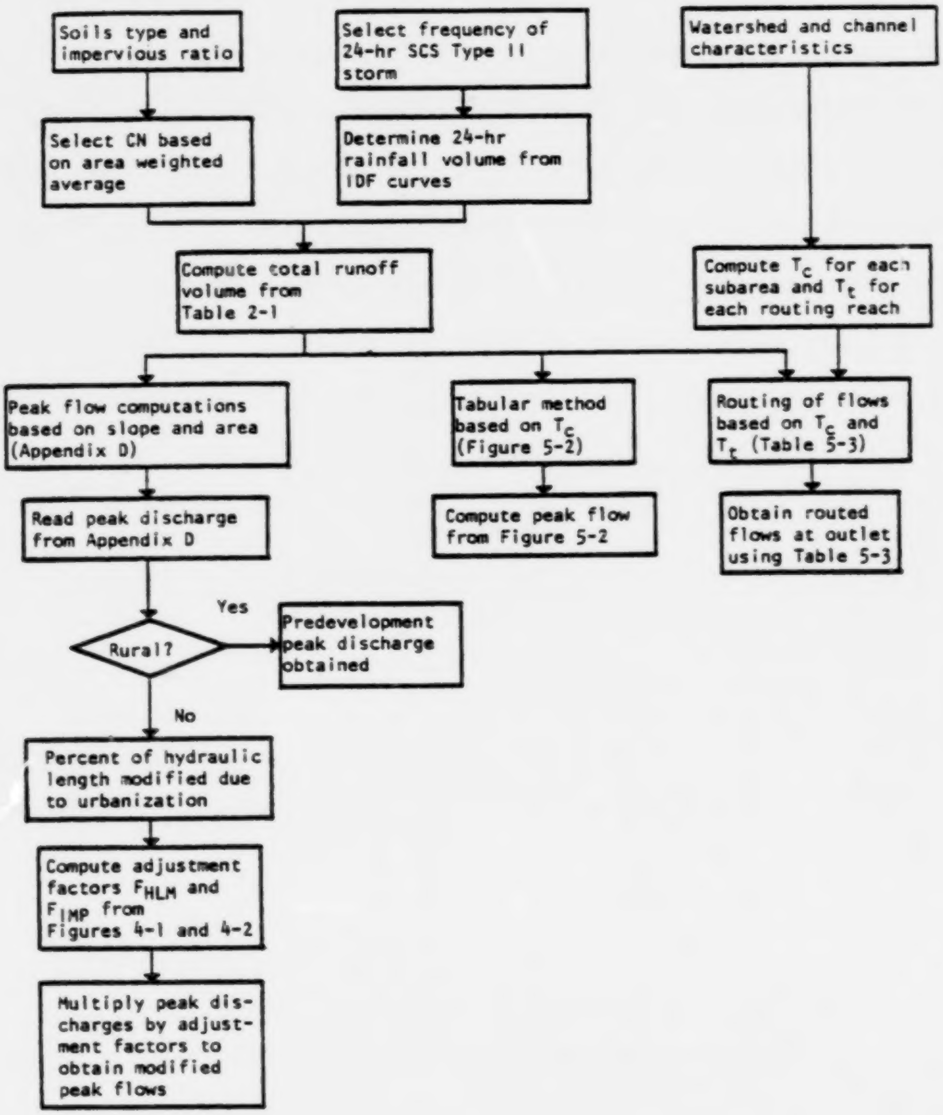

Soils type and impervious ratio
Select frequency of 24-hr SCS Type II storm
Watershed and channel characteristics
Select CN based on area weighted average
Determine 24-hr rainfall volume from IDF curves
Compute total runoff volume from Table 2-1
Compute T_c for each subarea and T_t for each routing reach
Peak flow computations based on slope and area (Appendix D)
Tabular method based on T_c (Figure 5-2)
Routing of flows based on T_c and T_t (Table 5-3)
Read peak discharge from Appendix D
Compute peak flow from Figure 5-2
Obtain routed flows at outlet using Table 5-3
Yes
Rural?
Predevelopment peak discharge obtained
No
Percent of hydraulic length modified due to urbanization
Compute adjustment factors F_{HLM} and F_{IMP} from Figures 4-1 and 4-2
Multiply peak discharges by adjustment factors to obtain modified peak flows

FIG. 1 FLOW CHART FOR SCS TR-55 RUNOFF COMPUTATIONS

short duration intense rainfalls are critical for storm sewer design, the meteorological input for SCS TR-55 method is not appropriate and may give lower peak flows.

(2) Runoff From Small Storms:

Total runoff volumes computed by SCS TR-55 for CN = 60 to CN = 90 for small rainfalls are shown in Figure 3. Runoff measurements (10) from 2 residential watersheds (see Table 1) are also shown in Figure 3. It can be seen that the runoff volumes obtained from real measurements are greater than runoff volume computed by TR-55 for CN = 90 which corresponds to an area with a high degree of imperviousness. It can be seen therefore that the procedures in SCS TR-55 which recommends the

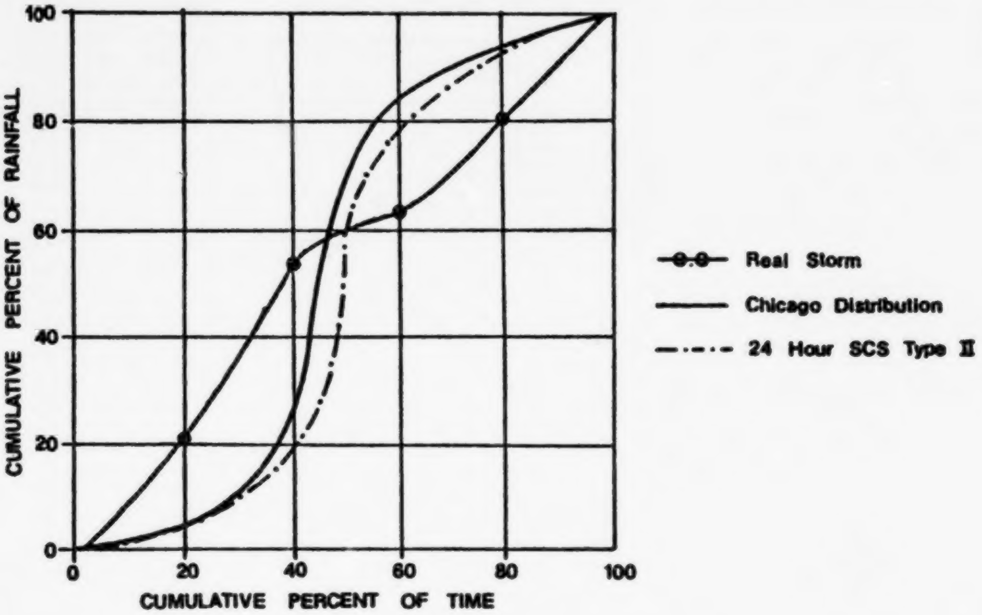


FIG. 2 RAINFALL TIME DISTRIBUTION CURVES FOR A REAL, CHICAGO, AND SCS TYPE II DISTRIBUTION

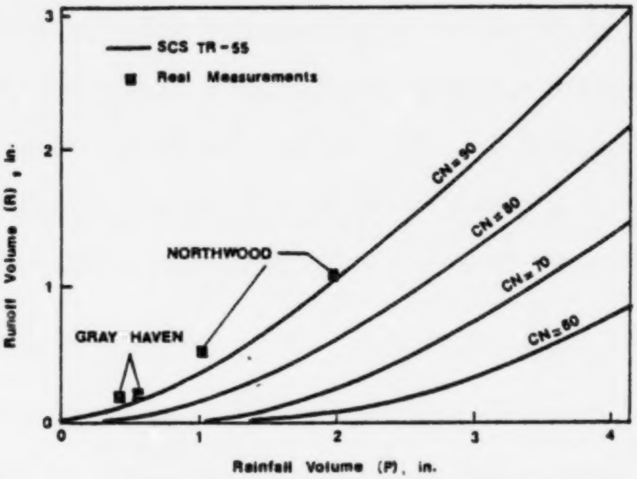


FIG. 3 RELATIONSHIP BETWEEN RAINFALL AND RUNOFF: TR-55 AND REAL MEASUREMENTS

TABLE 1

WATERSHED CHARACTERISTICS FOR REAL MEASUREMENTS

Name of Watershed	Drainage Area (acres)	Imp. Ratio (%)
Gray Haven	23.3	52
Northwood	47.4	68

selection of CN based on averaging requires the consideration of significant increases of CN for small rainfalls (say less than 3 in.). This finding was substantiated by a recent study (1) on several watersheds in Texas where calibrated CN's were found to be higher than those calculated from the manual.

An examination of the tables contained in TR-55 for the computation of runoff volume as a function of total rainfall and hydrologic soil complex number (CN) shows that for

CN varying from 75 to 85 (typical for Ontario conditions), the minimum rainfall required to generate (say) 0.1 in. of runoff would vary from 1.0 in. to 1.3 in. Thus a 1/2-year, 3 hour storm for Metro Toronto with a rainfall of 1.2 in. would yield 0.07 in. of runoff from a watershed with a CN = 75.

(3) Variations of Runoff Curve Numbers Within a Watershed:

According to the SCS TR-55 the composite CN for any watershed is calculated by means of a weighted average based on area. Runoff is not calculated separately for pervious and impervious areas as is done in several other models and as a result, the assumption discussed above becomes critical. This may be easily verified by means of a simple example. An urban residential watershed with an imperviousness ratio of 30 percent and 1/3 acre lot size and hydrologic soil group B would be assigned a CN = 72. The minimum precipitation required to generate a runoff of (say) 0.1 in. corresponding to this CN is approximately 1.5 in. For the same area however, the runoff computed by a model such as SWMM for the same rainfall would be approximately 0.5 in. It is also interesting to note that worked examples in the TR-55 report generally utilize storms of high return frequencies ranging from 1/50-yr. to 1/100-yr. with rainfall volumes of the order of 5.0 to 6.0 in., which do not correspond to conditions currently considered in most Canadian studies. The method was also used in the Maryland Pond Design Manual for low (1/2-yr) frequency storms (8).

A comparison of runoff volume coefficient computed by several procedures and degrees of imperviousness are shown in Table 2. These results are based on either simulations that have been carried out or computations based on information available in the literature. It can be seen that for small rainfalls the TR-55 tends to underestimate runoff volumes. The low C_v corresponding to Desbordes (2) measurements was probably caused by insufficient data for areas having low imperviousness ratios.

TABLE 2

COMPARISON OF RUNOFF VOLUME COEFFICIENTS

Imperviousness (%)	Runoff Volume Coefficient, C_v			
	MODELS			
	SWMM (Rainfall Volume = 1.22 in.)	STORM[2] (Annual Rainfall)	SCS TR-55 (Rainfall Volume = 1.22 in.)	DESBORDES[1] (Real Measurements Small Rainfalls)
30	0.29	0.38	0.14	0.18
60	0.57	0.60	0.32	0.49
90	0.86	0.83	0.63	0.79

[1] $C_v = 1.01 ((IMP/100 - 0.12))$ Reference 7

[2] In STORM used for individual storm events

(4) Adjustment of Computed Peak Flows:

One of the TR-55 methods (Appendix D of the manual) for computing and modifying computed peak discharges, includes two adjustment factors namely an imperviousness factor and a hydraulic length modification factor. Various users may work with different magnitudes for the last factor.

Other factors are watershed slope adjustment factor (WSAF), swamp and pond area adjustment factor (SPF) and watershed shape factor (WSF). These adjustment factors account for watershed slopes, swamp and ponded areas and shapes for situations other than those for which the graphs have been constructed.

Of the three adjustment factors in Appendix E in the release, watershed slope and shape factors are easy to apply once the average watershed slope and shape are known. However, the pond and swamp area adjustment factor requires some assessment of flow paths to determine whether a significant amount of the flow from the watershed passes through such areas and a study of topographic maps to locate such areas.

(5) Development of Tabular Discharges:

It can be shown that the relationship for peak discharges against T_c shown in Figure 5-2 and Table 5-3 in TR-55 can be plotted as a straight line on a log-log scale as shown in Figure 4 and can be represented by the following relationship:

$$q_p = \frac{3980}{T_c^{0.62}} \quad \text{--------------------(1)}$$

where q_p = peak flow in cfs/sq. mi/in. of runoff,

T_c = time of concentration.

For a watershed of drainage area of A sq. mi. and a 24-hr. rainfall of R in., the peak runoff rate is given by:

$$q_p' = \frac{3980AR}{T_c^{0.62}} \quad \text{--------------------(2)}$$

Equation 2 is analagous to a generalized relationship for peak runoff rate for a triangular hydrograph.

The tabular discharges in the technical release describe a unique hydrograph profile, viz., one obtained with a 24-hour SCS Type II storm. The routed hydrographs in terms

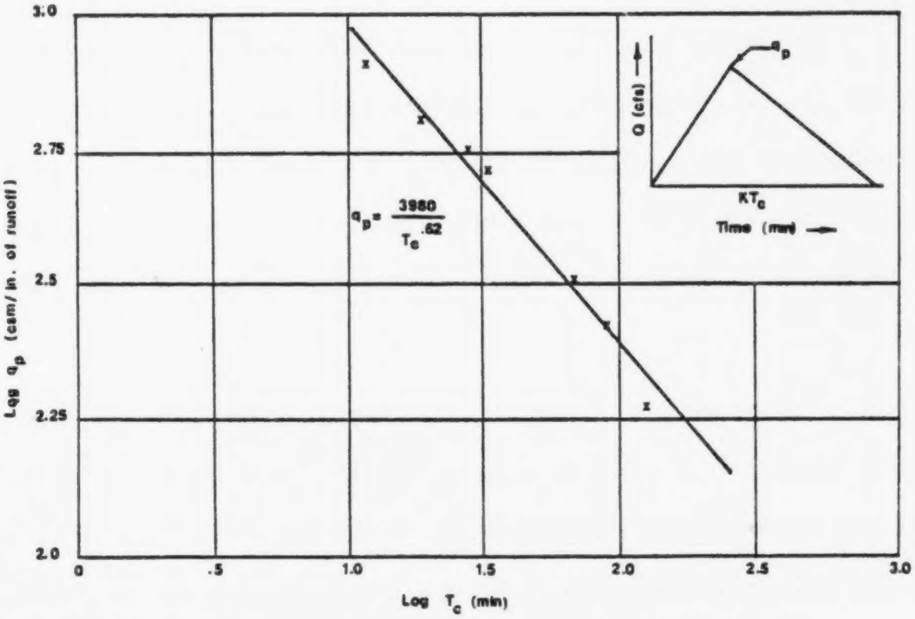


FIG. 4 RELATIONSHIP BETWEEN PEAK FLOW AND T_c: TR-55

of travel time (T_t) therefore cannot be used for any other storm profile or for other hydrographs.

(6) Alternative Procedures:

Although peak discharges from any given watershed can be obtained either in terms of time of concentration (T_c) or in terms of area, slope and CN, there is no clear recommendation as to the adequacy of the two procedures given in the manual.

This brief review indicates that the SCS TR-55 procedures cannot be applied for certain conditions for example short duration rainfalls, low imperviousness ratios etc. and that flows may be underestimated. So if peak flows are to be computed by TR-55, caution is required.

3.2 PEAK DISCHARGES: COMPARISON OF METHODS CONTAINED IN TR-55

There are two procedures for computing peak flows given in TR-55. One of the methods is based on the time of concentration for a watershed and the peak flows are given in terms of csm/in. while the other one is based on watershed area and slopes, and peak flows are given in terms of cfs./in. of runoff.

In order to compare the results obtained by the two methods, several watersheds ranging in size from 10 acres to 160 acres undergoing urbanization were considered. The watersheds in the predevelopment stage are considered to have uniform runoff characteristics and a CN = 75. For post development condition CN = 83 for a residential area of 1/4 acre lot size corresponding to an imperviousness ratio of 38% and hydrologic soil group (C). The 24-hr. rainfall for a 1/25 year storm for Oakville, Ontario (P_{24} = 3.31 in.) was used for the comparison.

The results and the watershed characteristics are shown in Table 3. It can be seen that different results are obtained by the two methods. The computations based on T_c give higher peak rates of runoff than those based on area and slopes. The percentage increase in peak flow using the computations based on T_c vary with area. For example, for 10 acres the increase is 8% whereas for 160 acres it is 55%. It therefore appears that the difference in peak flows would increase with the area. However, for urbanized conditions, the method based on area and slope give larger flows for areas up to 60 acres. For areas larger than 60 acres the method based on T_c computes flows which are once again larger but the percentage increase is much less. For example, for 160 acres the difference is only 22%.

3.3 COMPARISON WITH OTHER RUNOFF COMPUTATION MODELS

It is not possible to compare the results from discharge computations with SCS TR-55 against real measurements since it contains an implicit assumption; the use of a 24-hr. SCS Type II storm. However, it is possible to compare TR-55

TABLE 3

COMPARISON OF PEAK FLOW COMPUTATIONS BY TR-55 METHODS

	BASED ON APPENDICES D & E							BASED ON T_c		
	Watershed Area Acres (sq. mi.)	Factor Imperviousness	Factor Hydraulic Length Modified	W/S Slope Factor	W/S Shape Factor	q_p – Peak Flow per in. of runoff	Q_p – Peak Flow (cfs)	Time of* Conc. T_c (hr)	q_p – CSM per in. of runoff	Q_p – Peak Flow (cfs)
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Predevelopment Conditions, CN = 75, 24 Hour Rainfall = 3.31 in., Runoff (Table 2-1) = 1.18 in.	10(.016)	Not Required	Not Required	1.21	1.0	7.8	11.4	.35	650	12.3
	60(.094)	"	"	1.26	1.0	25.0	37.2	.58	470	52.0
	160(.25)	"	"	1.29	1.0	45.0	68.5	.85	360	106.2
Urbanized Conditions CN = 83, Runoff (Table 2-1) = 1.68 in.	10(.016)	1.17	1.43	1.21	1.0	9.0	30.6	.17	850	22.9
	60(.094)	1.17	1.43	1.26	1.0	32.0	113.2	.26	726	114.5
	160(.25)	1.17	1.43	1.29	1.0	58.0	210.0	.36	610	256.2

* T_c's are computed based on overland flow velocity of 2 ft/second and 15 min. inlet time for predevelopment conditions and a velocity 5 ft/second and 7.5 min. inlet time for urbanized conditions.

results with hydrograph models which have already been tested where the 24-hr. SCS storm may be simulated. Two models, namely HYMO for rural conditions (14) and SWMM for urbanized conditions (9) were selected for this purpose.

For rural conditions, the same hypothetical watersheds (10 acres to 160 acres) were used for which TR-55 computations were performed. An average CN of 75 was used to reflect low imperviousness ratios. This is based on an earlier study (15) where it was found that infiltration losses with HYMO and SWMM (with default values) are close for imperviousness CN = 98 and pervious CN = 75. Table 4 shows the peak flows computed by the two models. It is seen that although for very small areas (up to 10 acres) and rural conditions, the difference in flows is small, the difference increases sharply for larger areas. For example, for an area of 160 acres the difference is as high as 55 percent.

For urbanized conditions, where 38 percent of the watershed is impervious, peak rates of runoff and hydrographs computed by TR-55 were compared with those simulated by SWMM. These results are shown on Table 4 and Figure 5. The following may be observed:

(1) The peak rates of runoff computed by TR-55 (T_c method) is consistently lower than those computed by SWMM up to 160 acres;

TABLE 4

COMPARISON OF PEAK FLOWS: SCS TR-55 (BASED ON T_c), HYMO AND SWMM

Watershed Area (acres)	PEAK FLOWS (CFS)			
	PREDEVELOPMENT CONDITIONS		POST DEVELOPMENT CONDITIONS	
	SCS TR-55 (Runoff = 1.18 in.)	HYMO (Runoff = 1.10 in.)	SCS TR-55 (Runoff = 1.68 in.)	SWMM* (Runoff = 1.66 in.)
(1)	(2)	(3)		
10	12.3	13.5	22.9	27.9
60	52.0	71.9	114.5	132.8
160	106.2	166.9	256.2	290.6

---

* Imperviousness ratio = 38 percent.

(2) For small areas up to 60 acres the difference is of the order of 20 percent and appears to decrease for larger areas.

It may be mentioned however, that some of the input data to TR-55 (CN for example) can be modified in order to obtain flows which would be much closer to those simulated by SWMM.

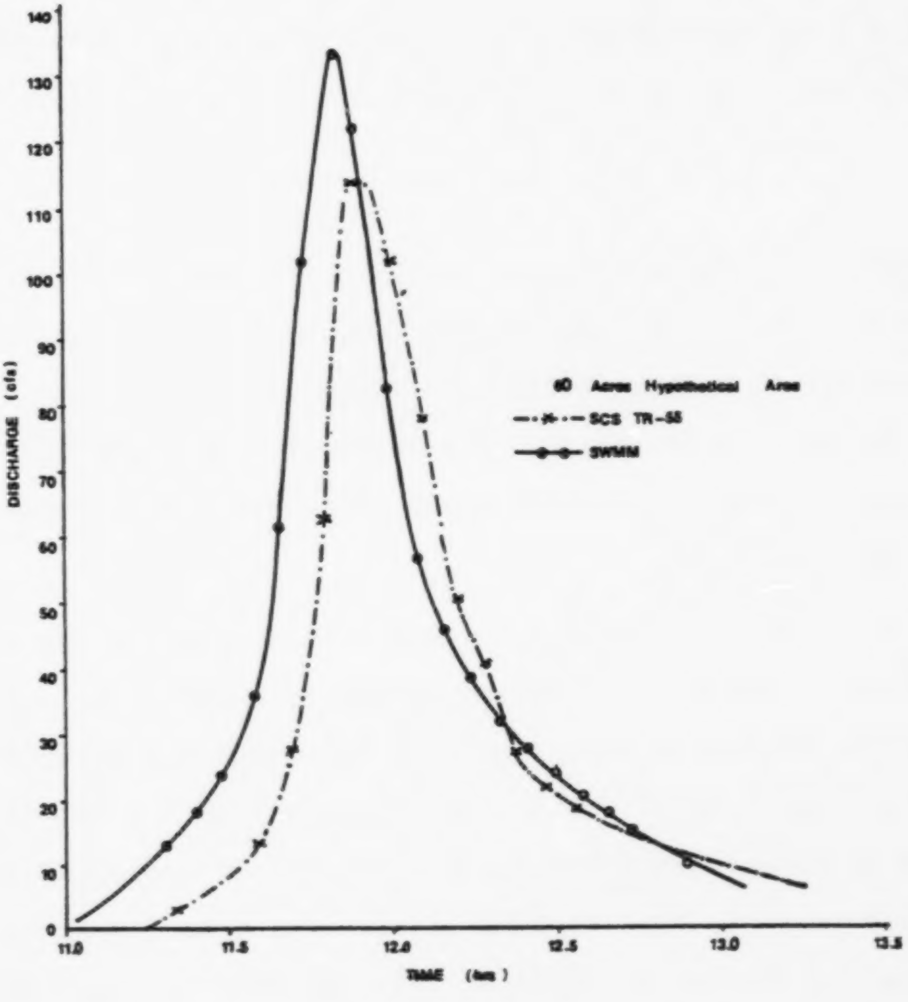


FIG.5 COMPARISON OF HYDROGRAPHS : SCS TR-55 (TABULAR METHOD) AND SWMM

3.4 CHANNEL ROUTING

The tables contained in the TR-55 manual for routing runoff hydrographs through stream channels are based on the convex method (13). The routing equation in the convex method is:

$$O_2 = (1-C)\ O_1 + C\ I_1 \text{ -------------------------(3)}$$

where I is the inflow to the reach, O is the outflow from the reach and C is the storage coefficient given by

$$C = \Delta t/k \text{ ----------------------------------(4)}$$

where Δt is the wave travel time and also the required routing interval, k is the reach travel time for a selected steady flow discharge of a water particle through the reach; subscripts 1 and 2 refer to the beginning and end of time interval. More details on the convex method can be found in the SCS National Handbook (11).

For the purpose of assessing routing by the TR-55 tables, a hypothetical system formed by a watershed and a channel (Figure 6) was used. The watershed has an area of 0.4 sq. mi. and CN = 80. For a rainfall of 6.0 inches and time of concentration of 1.0 hr, the runoff hydrograph obtained from the TR-55 tables (for travel time = 0.0) is routed along the channel by means of: (a) the TR-55 tables, (b) the convex method, and (c) the WRE-TRANSPORT Model (4,6), a hydraulic routing model used extensively in Canada and the U.S. It was shown by the first and third authors that routing by the WRE-TRANSPORT model is accurate when compared with the method of characteristics (5). The routed hydrographs by the three methods are compared in Table 5 and shown in Figure 6 at two locations: 15,643 ft. and 29,683 ft. downstream of the watershed outlet.

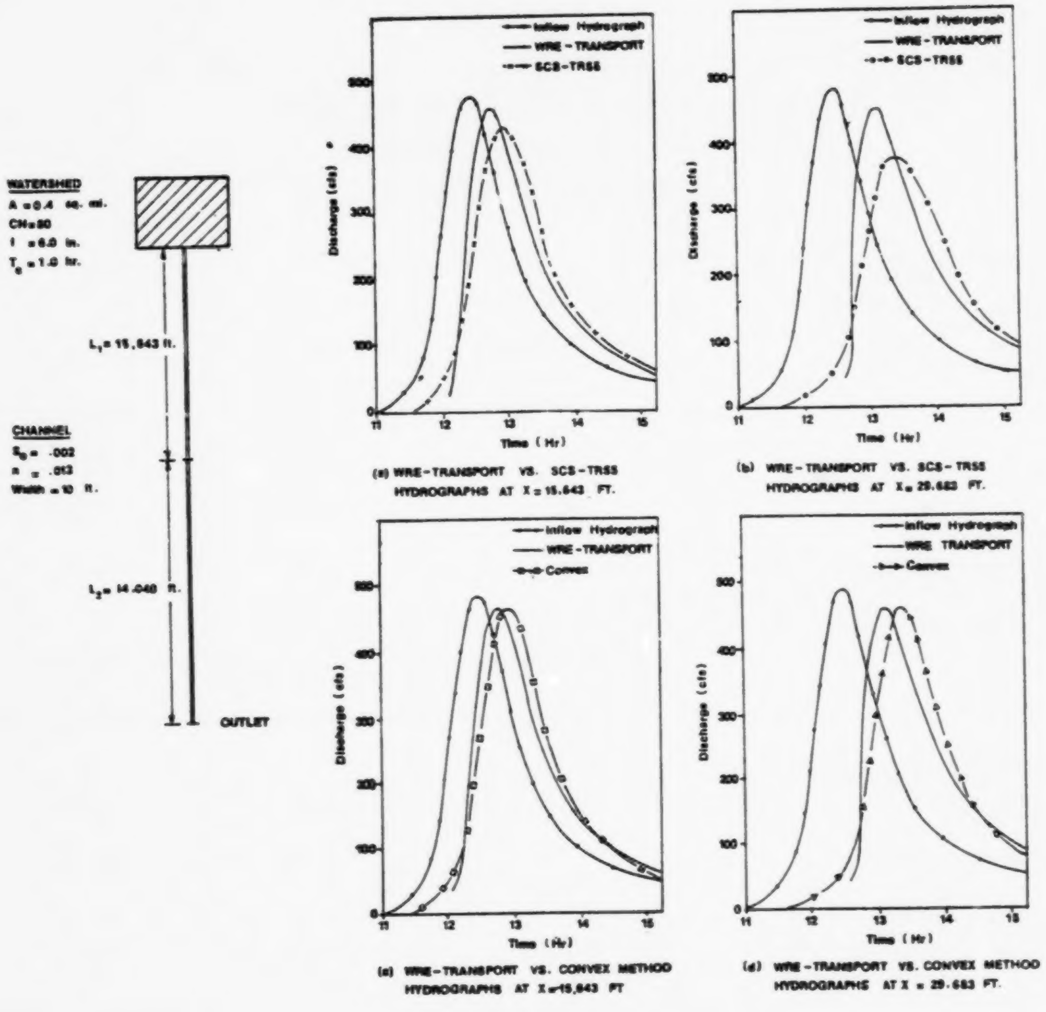
TABLE 5

ASSESSMENT OF ROUTING BY THE TR-55 TABLES - COMPARISON WITH THE CONVEX METHOD AND THE WRE-TRANSPORT MODEL

Hydrograph Location	Routing Method	WRE-Transport	Convex	SCS TR-55
x = 15,643 ft	Peak Discharge Qp (cfs)	461	465	429
	Error in Qp*	-	0.9%	6.9%
	Time to Peak (hr)	12.75	12.91	12.90
x = 29,683 ft	Peak Discharge Qp (cfs)	450	453	396
	Error in Qp*	-	0.7%	12.0%
	Time to Peak (hr)	13.10	13.38	13.50

* In comparison with WRE-Transport Model

It can be seen that the TR-55 routing tables tend to underestimate flows. The error increases with travel time, that is with the length of channel. A direct application of the convex method, on the other hand, gives more accurate results.



(a) WRE-TRANSPORT VS. SCS-TR55 HYDROGRAPHS AT X = 15,643 FT.

(b) WRE-TRANSPORT VS. SCS-TR55 HYDROGRAPHS AT X = 29,683 FT.

(c) WRE-TRANSPORT VS. CONVEX METHOD HYDROGRAPHS AT X = 15,643 FT

(d) WRE-TRANSPORT VS. CONVEX METHOD HYDROGRAPHS AT X = 29,683 FT.

FIG. 6 COMPARISON OF CHANNEL ROUTING BY THE WRE - TRANSPORT MODEL, SCS - TR55 TABLES AND THE CONVEX METHOD

Although a complete assessment of these tables would require more testing under a variety of conditions, there is an apparent major limitation of the tables, that is the use of a hydrograph profile corresponding to 24-hr. Type II storm.

The routing tables may be attractive because of their simplicity. The above test shows however that the convex method, which is as simple to use as the TR-55 tables, is more accurate and does not have any limitation regarding the hydrograph profile.

SUMMARY AND CONCLUSIONS

(1) Users of the SCS TR-55 procedures should be aware of the limitations introduced by the meteorological input assumptions and the methodology for calculation of infiltration and other losses, viz.;

  (a) a particular distribution and a specific (24-hours) duration design storm;

  (b) weighted averaging of curve numbers (CN) for pervious and impervious areas which may lead to overestimation of rainfall losses corresponding to 1/2 to 1/10-year storms.

(2) The two procedures for peak discharge computations in TR-55 lead to different results.

(3) The second method giving peak flows in terms of areas and slopes seems to significantly underestimate peak flows.

(4) For the same meteorological input and rainfall losses, simulations carried out by the "time of concentration" method underestimated the peak flows for areas up to 160 acres as compared to SWMM.

(5) The tabular procedures in the SCS TR-55 for routing runoff hydrographs through stream channels seem to underestimate the flows and can be used only in conjunction with a 24-hr., Type II storm.

(6) Direct use of the convex method seems to be more accurate and can be used for any hydrograph profile.

FINAL REMARKS

The philosophy of the TR-55 seems to be that stormwater management computations can be carried out by a step by step method without giving the practitioners too much information regarding the limitations of the method, testing with meansurements or other models etc. while such an approach makes the method relatively attractive, it may lead to considerable errors. On the other hand if properly applied some of its features are quite attractive.

The main purpose of this paper is to invite discussion regarding the use of such "black box" approaches whether it applies to desk top or computerized techniques. It is also felt that agencies in charge with implementation of SWM and flood control studies should carefully review methodologies and be aware of their limitations.

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A SIMPLIFIED STORMWATER QUANITY AND QUALITY MODEL

By

S. Sarikelle[1] and Yu-Tang Chuang[2]

INTRODUCTION

Urban stormwater management has drawn much attention in recent years due to problems associated with quality of storm runoff and its detrimental effects on receiving waters. Studies have shown that pollution carried by stormwater discharging untreated and highly polluted street washings into receiving waters exceed those discharges from secondary municipal treatment plant effluents (1,2,3). Indeed, in many cases, the quality of receiving water is more likely to be governed by waste produced in urban areas which is flushed away by runoff.

Due to the very complex nature of the urban rainfall-runoff quality process, it is recognized that the utilization of simulation models provide an efficient means for the investigation of the various aspects in urban drainage systems. Over the years, various computerized mathematical models such as U.S. E.P.A. Stormwater Management Model, University of Cincinnati Urban Runoff Model, and Hydrocomp Simulation Program, etc. (4) have been developed. Most of the existing models, however, have complicated structure, require extensive input data, and do not incorporate the qualitative aspect of the urban runoff phenomena efficiently. Therefore it is felt that there is a need to develop a quantity and quality simulation model which utilizes simplified rainfall runoff relationship and yields results which are comparable to those obtained by the more comprehensive and complex models.

The study presented herein summarizes the development of a quantity and quality urban runoff simulation model with simple input requirements to be used in the planning and analysis of stormwater systems. The model entitled MLSURM, an acronym for a Modified Linearized Subhydrographs Urban Runoff Model, consists of a quantity submodel and a quality submodel.

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Quantity Submodel

The quantity submodel basically represents the simulation of runoff for a given area which results from a rainfall event. The method is based on a simplified concept (5,6) where hydrographs are generated for each subarea in accordance with the duration of rain and the time of concentration. Three cases of linearized subhydrographs are assumed for each subcatchment.

In Case I, $t_r = t_c$; that is, storm duration equals the time of concentration for the subbasin. As shown in Figure 1, the peak runoff occurs when the total flow from the subbasin contributes to the inlet the peak runoff rate is defined by:

$$q_p = C\ i\ A \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where:

q_p = peak runoff rate (ft^3/sec)

C = runoff coefficient

i = intensity of rainfall (inches/hour)

A = area of the subbasin (acres)

The runoff rate of the rising and receding limbs are determined by assuming a linear relationship between the runoff and time although it is known that they are of a nonlinear nature. The linear assumption is realistic since the subbasins are small the nonlinear variation of the subhydrograph can also be assumed to be small. Therefore, for $t \leq t_r$, the runoff rate q_t at time t is given by:

$$q_t = C\ i\ A\ \frac{t}{t_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

For $t > t_r$, the runoff rate is calculated by:

$$q_t = C\ i\ A\ \left(\frac{t_r + t_c - t}{t_c}\right). \quad . \quad . \quad . \quad . \quad (3)$$

The time base of the subhydrograph t_b is computed by:

$$t_b = t_r + t_c = 2t_r \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The volume of runoff V resulting from the storm is then calculated by:

$$V = C\ i\ A\ t_r \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

In Case II, the storm duration is assumed to be greater than the time of concentration for the subbasin, that is $t_r > t_c$. Thus, after a time period equal to the time of concentration, the peak runoff rate is reached and remains constant

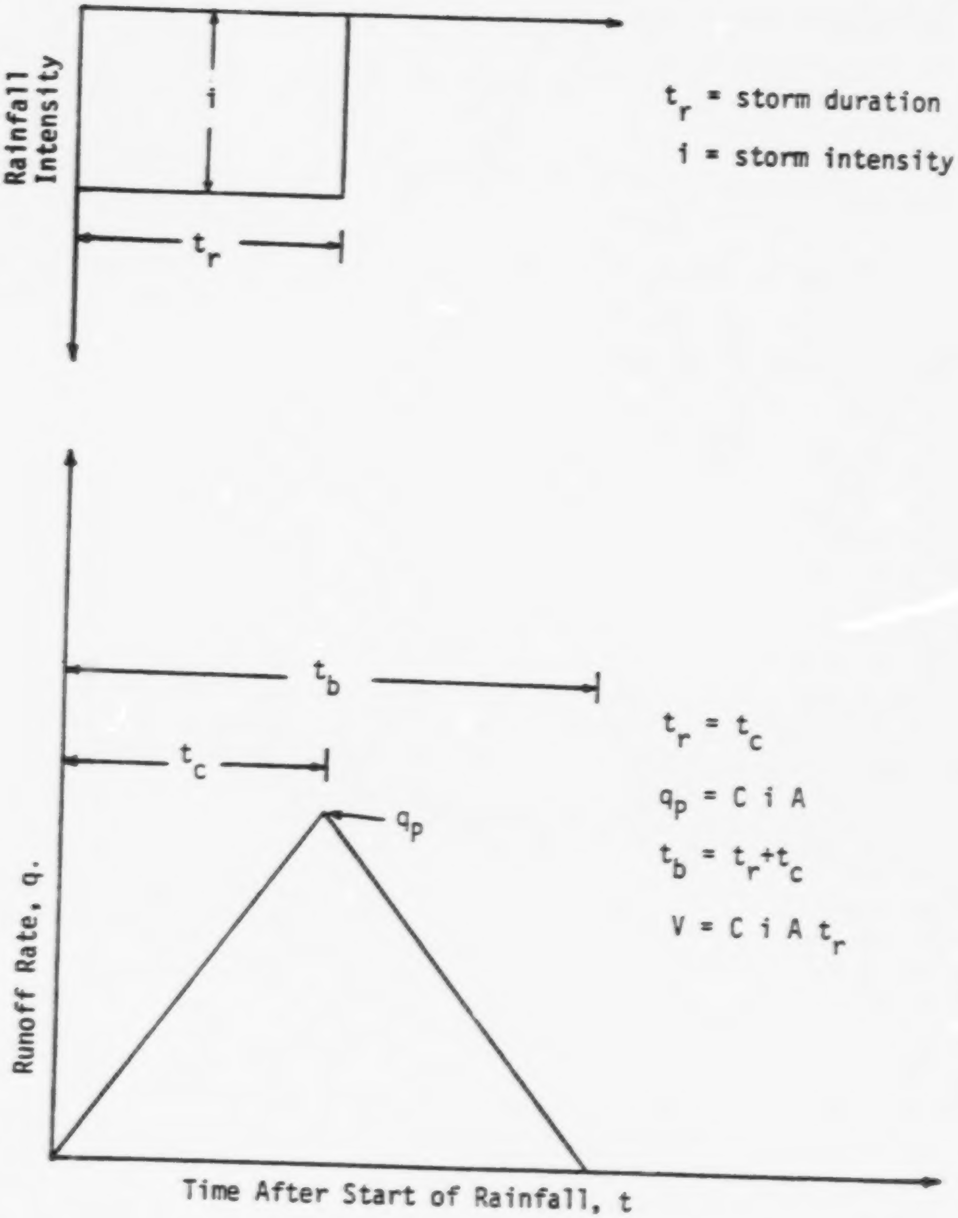

Rainfall Intensity
i
t_r
t_r = storm duration
i = storm intensity
Runoff Rate, q.
t_b
t_c
q_p
$t_r = t_c$
$q_p = C\ i\ A$
$t_b = t_r + t_c$
$V = C\ i\ A\ t_r$
Time After Start of Rainfall, t

Figure 1 Case I Linearized Subhydrograph

until the rainfall stops. Then, the runoff recedes to zero in a time period which equals to t_c. As shown in Figure 2, a trapezoidal shape subhydrograph is proposed. The runoff rate of the subhydrograph is given by:

$$\text{For } t < t_c,\ q_t = C\ i\ A\ \frac{t}{t_c} \tag{6}$$

$$\text{For } t_c \leq t \leq t,\ q_t = C\ i\ A = q_p \tag{7}$$

$$\text{For } t > t_r,\ q_t = C\ i\ A\ \left(\frac{t_r + t_c - t}{t_c}\right) \tag{8}$$

The time base of the subhydrograph is given by:

$$t_b = t_r + t_c \tag{9}$$

The runoff volume is then computed by:

$$V = C\ i\ A\ t_r \tag{10}$$

In Case III, $t_r < t_c$, that is the time of concentration for the subbasin is greater than the storm duration. Thus, the equilibrium runoff rate is not reached when the storm ceases. The peak runoff rate is calculated by:

$$q_p = C\ i\ A\ \frac{t_r}{t_c} \tag{11}$$

After the storm stops, the runoff rate will remain constant until a time t_p is reached, then the runoff recedes to zero (7). The value of t_p depends on the values of t_r and t_c and is given by

$$t_p = 0.4\ t_r + 0.6\ t_c \tag{12}$$

Therefore, the runoff rate of the subhydrograph is given by

$$\text{For } t < t_r,\ q_t = C\ i\ A\ \frac{t}{t_c} \tag{13}$$

$$\text{For } t_r \leq t \leq t_p,\ q_t = C\ i\ A\ \frac{t_r}{t_c} \tag{14}$$

$$\text{For } t > t_p,\ q_t = C\ i\ A\ \frac{t_r}{t_c}\ \left(\frac{t_b - t}{t_b - t_p}\right) \tag{15}$$

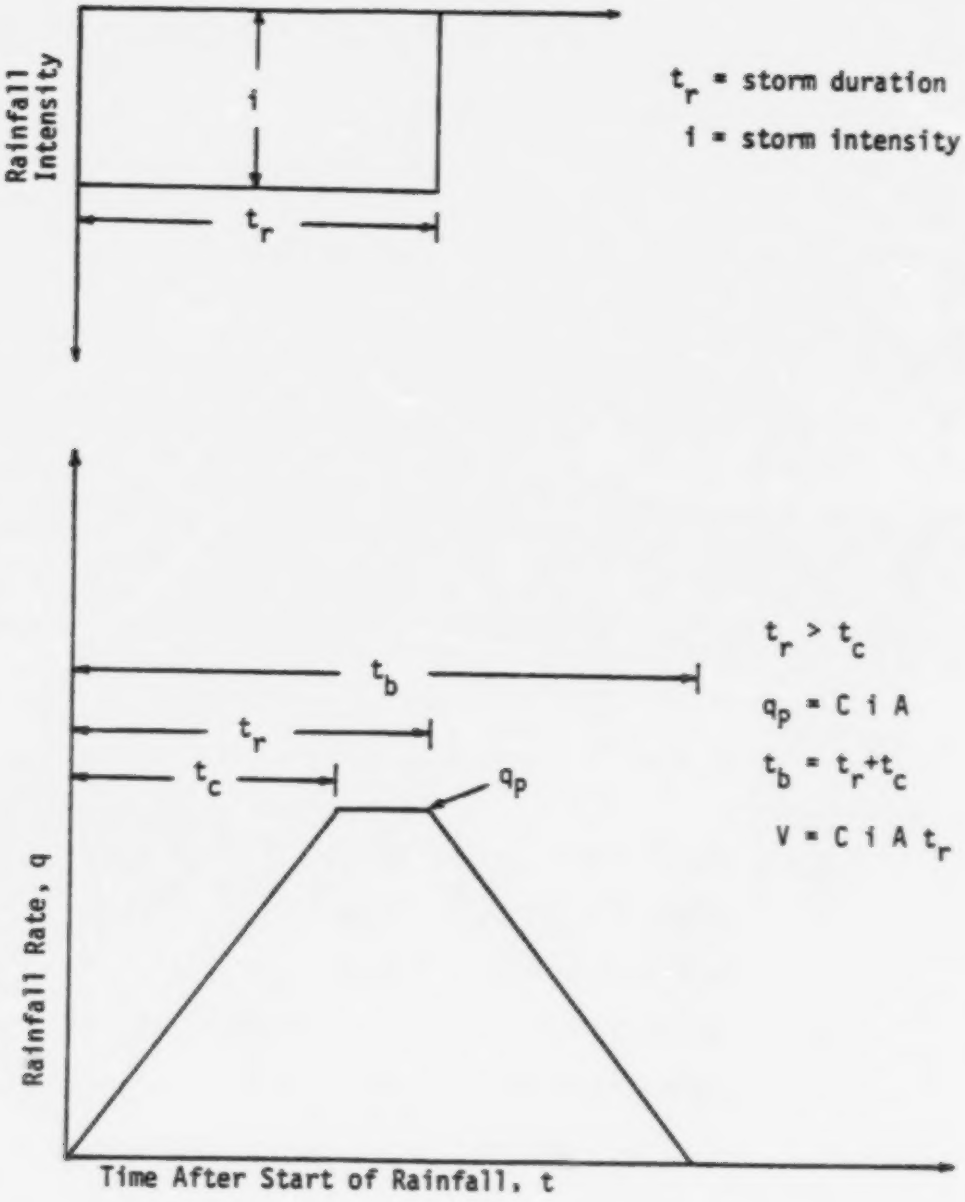

Rainfall Intensity
i
t_r
t_r = storm duration
i = storm intensity
t_b
t_r
t_c
q_p
$t_r > t_c$
$q_p = C\ i\ A$
$t_b = t_r + t_c$
$V = C\ i\ A\ t_r$
Rainfall Rate, q
Time After Start of Rainfall, t

Figure 2 Case II Linearized Subhydrograph

where:

$$t_b = 0.6\, t_r + 1.4\, t_c \quad . \quad . \quad . \quad . \quad . \, (16)$$

The runoff volume is determined by:

$$V = C\, i\, A\, t_r \quad . \quad . \quad . \quad . \quad . \quad . \, (17)$$

A schematic representation of this case is shown in Figure 3.

The runoff coefficient C represents the abstractions, or "losses", between rainfall and runoff generated from a particular subcatchment. It is noted that the abstractions decrease in magnitude as the duration of the storm increases. In the development of MLSURM model, Hoad's runoff coefficients (8) as shown in Figure 4 are adopted.

The time of concentraion for a subbasin is equivalent to the inlet time, this is the time required for the surface runoff to flow from the most remote point of the subbasin to the inlet of the subbasin. Various methods of estimating time of concentration for a drainage area have been proposed. For overland flow, a formula based on kinematic wave theory (9) was used in the model. The equation is given by the following relationship:

$$t_c = 0.928 \frac{L^{0.6} \quad N^{0.6}}{i^{0.4} \quad S^{0.3}} \quad . \quad . \quad . \quad . \quad . \quad . \, (18)$$

where

t_c = time of concentration (minutes)

L = length of overland flow (feet)

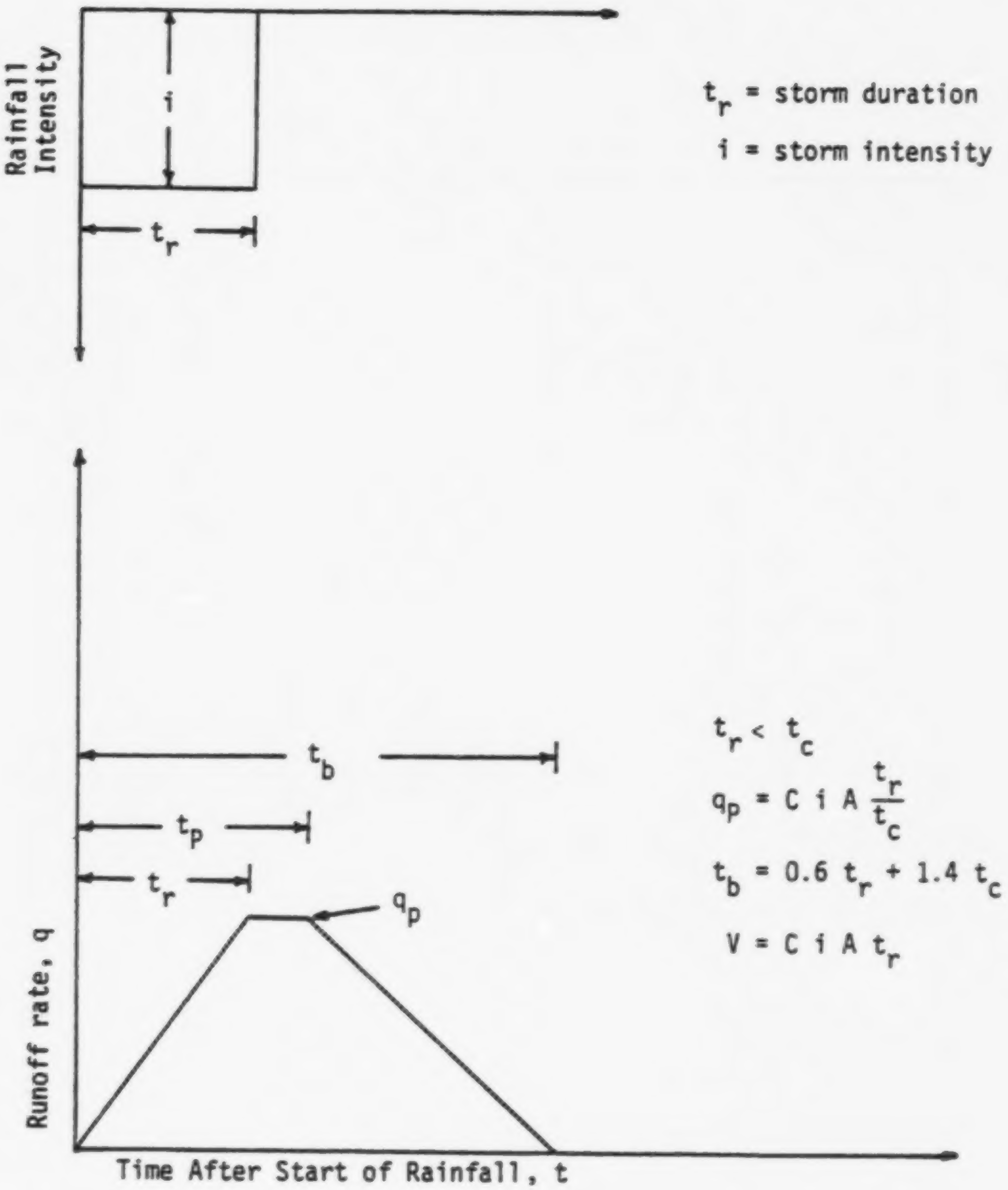
N = Manning's roughness coefficient

i = intensity of excess rainfall (inches/hour)

S = average overland slope (foot per foot)

For large subbasins with street curbs, the time of concentration is obtained by summing the overland time and the time of travel in the gutter. The overland time for the impervious area or the pervious area is determined by using the distance from upstream portion of the impervious area or pervious area to the gutter as overland length. Based on overland time, an initial overland hydrograph contributing to the gutter is obtained by the linearized subhydrograph procedure. Gutter flow travel time computed using the initial overland hydrograph is then included in the time of concentration to develop the inlet hydrograph.

In application where sewered areas are considered, the hydrograph resulting from each subcatchment must be routed through the sewer system to obtain the out-



Rainfall Intensity
i
t_r
t_b
t_p
t_r
q_p
Runoff rate, q
Time After Start of Rainfall, t

t_r = storm duration

i = storm intensity

$t_r < t_c$

$$q_p = C\ i\ A\ \frac{t_r}{t_c}$$

$$t_b = 0.6\ t_r + 1.4\ t_c$$

$$V = C\ i\ A\ t_r$$

Figure 3 Case III Linearized Subhydrograph

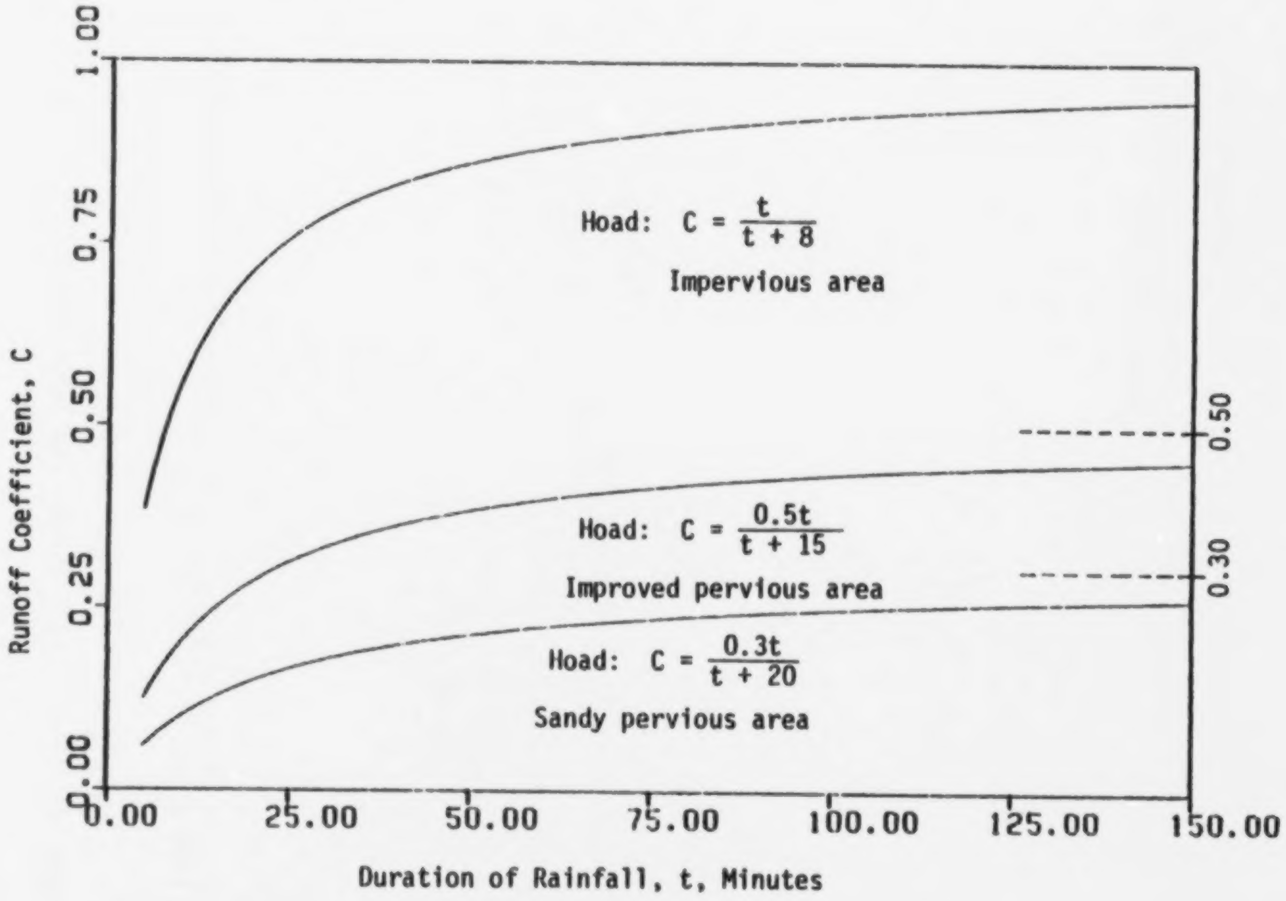


Figure 4 Runoff Coefficients vs Duration of Rainfall and Area Characteristics (21)

fall hydrograph. The "time-offset" routing procedure developed by Tholin (10) is utilized. The travel time in the sewer is determined by assuming uniform flow condition in the sewer pipe. The inflow hydrograph is then offset according to the travel time. In this fashion the routed hydrographs are then utilized to develop system hydrographs at various points in the sewer network.

Quality Submodel

The development of the quality submodel centers on the determination of pollution levels in the stormwater. Surface pollutants consist of street litter and dustfall that accumulate on the ground and street surfaces prior to a storm. When the storm occurs, the accumulated materials on the surface are dissolved by rain. As rainfall continues, surface runoff begins to wash off the pollutants. The impact of the raindrops on the relatively rough surfaces provides a high level of turbulence which tends to accelerate the pollutant removal process.

Keeping in mind the complexity of the pollutant removal process, an attempt is made to develop a relationship that would be able to describe such phenomenon and aid in the proper simulation of the pollutant wash-off from the basin surfaces. The relation is given as follows:

$$-\frac{dP}{dt} = k\,P\quad q\sqrt{\frac{S}{y}} \quad . \quad . \quad . \quad . \quad . \qquad (19)$$

where:

$\frac{dP}{dt}$ = rate of washout of pollutant (pounds/hour)

P = amount of pollutant remaining on the surface (pounds)

q = rate of runoff (inches/hour)

S = overland slope (ft/ft)

y = depth of flow (ft)

K = proportionality constant

Integrating equation (19) yields

$$P_{t+\Delta t} = P_t \; e^{-K\sqrt{S}\,\frac{\Delta V}{\sqrt{y}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad .(20)$$

where:

$P_{t+\Delta t}$ = amount of pollutant remaining on the surface at time $t+\Delta t$ (pounds)

P_t = amount of pollutant on the surface at time t (pounds)

Δt = time interval (hour)

Δv = incremental runoff volume during Δt (inches)

y = depth of runoff flow (ft)
S = overland slope (ft/ft)
K = proportionality constant

Thus, the rate of surface pollutant washout M_s in pounds per hour is:

$$M_s = (P_t - P_{t+\Delta t})/\ \Delta t \ = P_t\ (1 - e^{-K\sqrt{S}\ \frac{\Delta V}{\sqrt{y}}})/\ \Delta t \quad . \quad . \quad . \ (21)$$

Equation (21) gives the amount of surface runoff pollutograph corresponding to the inlet hydrograph. The equation is applied successively, the value of $P_{t+\Delta t}$ which is determined at the end of the current interval becomes the value of P_t at the beginning of the next interval.

Equation (21) attempts to model the surface pollutant removal process with the following restrictions:

1. For catchment subjected to identical rainfall input, the one with steeper overland slope would result in faster pollutant removal rate.

2. For a particular catchment where overland slope is fixed, the motive force of washing out the pollutant then depends on the ratio, $\Delta V/\sqrt{y}$. The higher this ratio, the more significant the motive force becomes in washing out the surface pollutant. This relationship indicates the effect of incremental runoff volume and the effect of the depth upon pollutant removal efficiency.

In applications to watersheds with sewer systems, the pollutographs entering into the sewer must be routed to yield the outfall pollutograph. The routing procedure similar to that used in the SWMM model (11) is incorporated in the model developed herein.

Model Description

The model structure basically follows the concepts presented in the preceding sections. A generalized flow chart that shows the basic quantity and quality simulation algorithm is shown in Figure 5. The model is divided into a main driving program and 43 subprograms. The object-time dimension feature of FORTRAN IV is utilized to allocate the adjustable dimension variables into a single one-dimensional array. The size of the array is declared in the main program. Advantages in the program's memory storage allocation make it compatible with standard compilers so that computer systems comparable to that of the IBM 370 or UNIVAC 1108 can be used.

Application of Model

The model was applied to two typical urban drainage areas with measured runoff data corresponding to various gaged storm events.

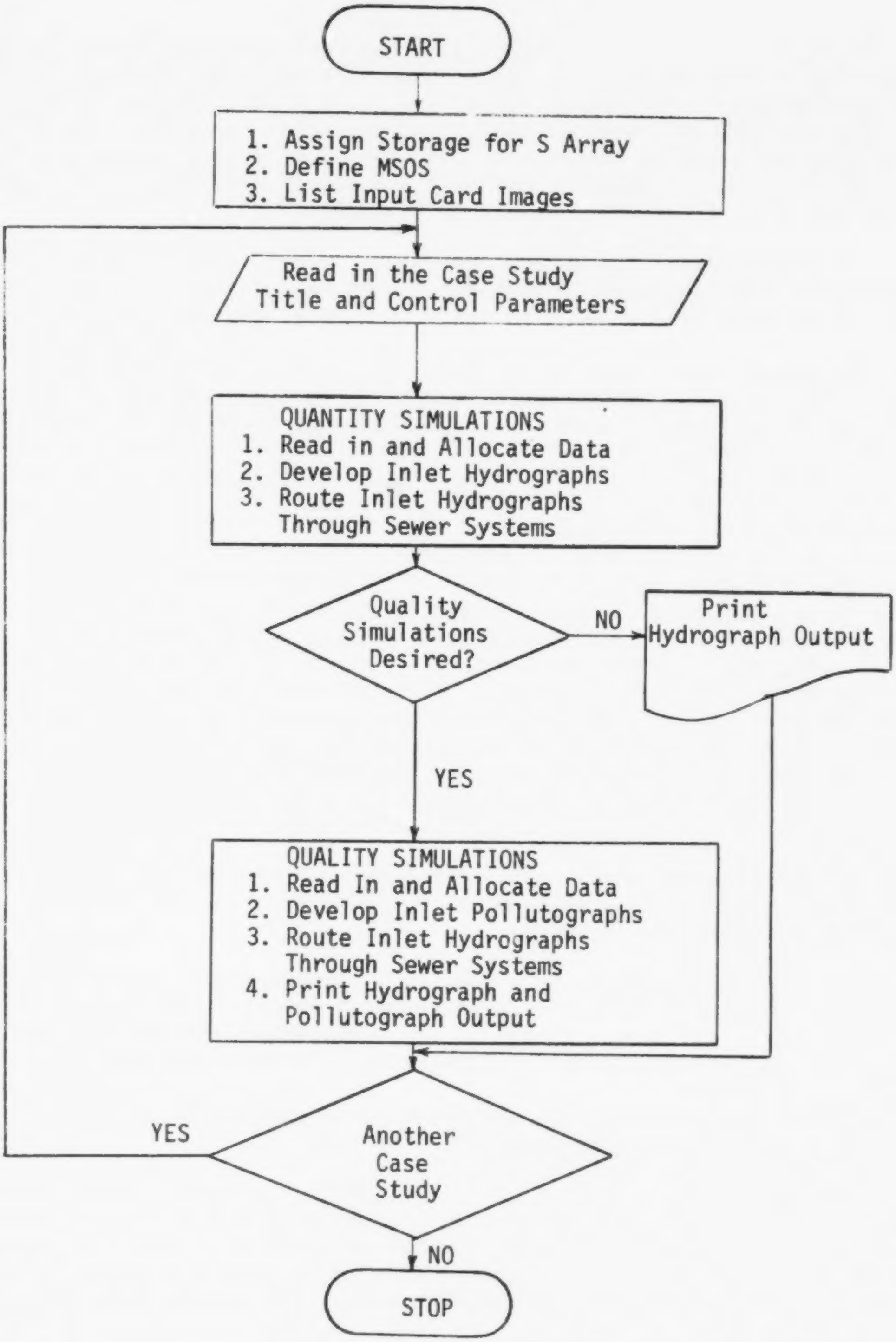

START
1. Assign Storage for S Array
2. Define MSOS
3. List Input Card Images
Read in the Case Study Title and Control Parameters
QUANTITY SIMULATIONS
1. Read in and Allocate Data
2. Develop Inlet Hydrographs
3. Route Inlet Hydrographs Through Sewer Systems
Quality Simulations Desired?
NO
Print Hydrograph Output
YES
QUALITY SIMULATIONS
1. Read In and Allocate Data
2. Develop Inlet Pollutographs
3. Route Inlet Hydrographs Through Sewer Systems
4. Print Hydrograph and Pollutograph Output
YES
Another Case Study
NO
STOP

Figure 5 Generalized Flowchart of MLSURM Model

The first drainage area selected is the Oakdale Avenue Basin located in a residential section about six miles northwest of downtown Chicago. The 12.9 acre basin is composed of 7.05 acres of pervious area and 5.85 acres of impervious area. Due to the small size of the basin, the entire drainage area is treated as a single catchment in the computer simulation instead of being subdivided into individual subcatchments. Rainfall and runoff data have been recorded since 1959 by the City of Chicago's Bureau of Engineering (12).

The simulation of the storm occuring on July 2, 1960 is shown in Figure 6. In comparing the simulated hydrographs to the recorded hydrographs, the time to peaks and the peak discharges from the MLSURM model yielded fairly good agreement.

The second application of the model is the Mortimer Avenue Basin in Toronto, Canada. The basin is located about four miles to the northeast of downtown Toronto in the Borough of East York. The 383 acre catchment has been subdivided into 33 subcatchments. A study by M.M.Dillon Limited Consulting Engineers (13) generated gaged rainfall data and runoff data for this basin. The runoff hydrographs, the suspended solid pollutgraphs and levels of BOD were also determined. However, no precise information about the amount of pollutant accumulated on the surface prior to a storm was available. Therefore the accumulation rates of the dust and dirt, pollutant content of the dust and dirt,and the pollutant removal rate constant K were calibrated by comparing the simulation results against recorded values. The calibrated value of these parameters are shown in Table I. The simulation results are presented in Figures 7 and 8. As can be seen from the results, both the peak discharges and the time synchronization of the MLSURM simulated hydrographs are in good agreement with the recorded hydrographs. In the simulation of pollutographs, the results obtained by the MLSURM model also compare well to the recorded pollutographs.

Conclusions

An urban runoff model entitled MLSURM is developed for the simulation of storm water quantity and quality. The following conclusions are reached based on the test application of the model:

1. The model simulated stormwater hydrograph well for small urban catchments with little calibration effort. The successful simulation of hydrographs for large watersheds is also achieved if the watersheds are subdivided into small catchments.

2. Satisfactory pollutograph simulations were also obtained by using calibrated parameter values.

The model may be used as an alternative to the more comprehensive and complex models in the planning and analysis of stormwater systems. It incorporates simple but physically realistic parameters in the simulation of runoff of stormwater systems. However, the model should be applied to more watersheds including urban, suburban, and rural areas where runoff quantity and quality data are available.

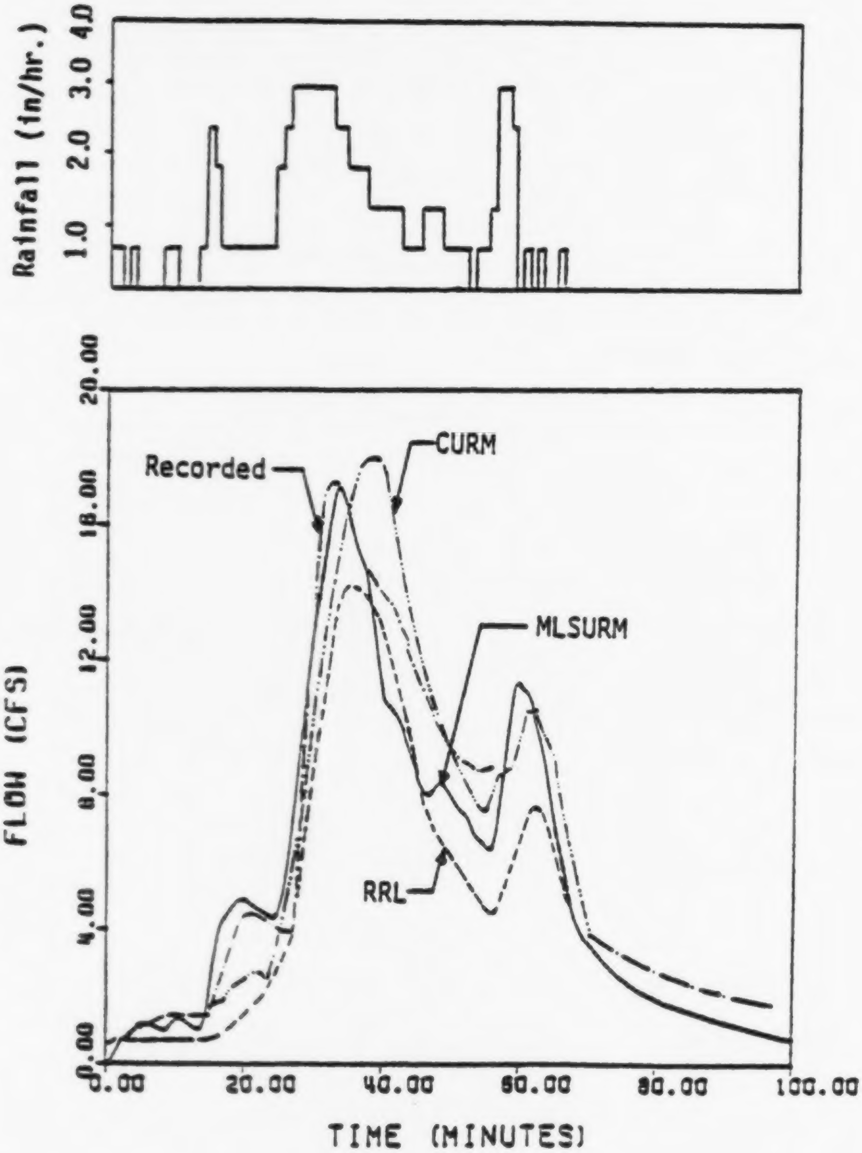


Figure 6 Results from Oakdale Avenue Basin, Chicago Storm of July 2, 1960

Table I Calibrated Accumulation Rates of Dust and Dirt, Pollutant Contents in Dust and Dirt, Pollutant Removal Constant K

Land Use	Accumulation Rate of Dust and Dirt (Pounds/Day/100 ft - curb)
Single family, residential	1.4
Commercial	6.6
Undeveloped or Park	3.0

Land Use	Milligram pollutant/gram Dust and Dirt	
	Suspended Solids	BOD
Single family, residential	160	35.
Commercial	160	30.8
Undeveloped or Park	160	35.

Pollutant Removal	Suspended Solids	BOD
Constant K	10.	15.

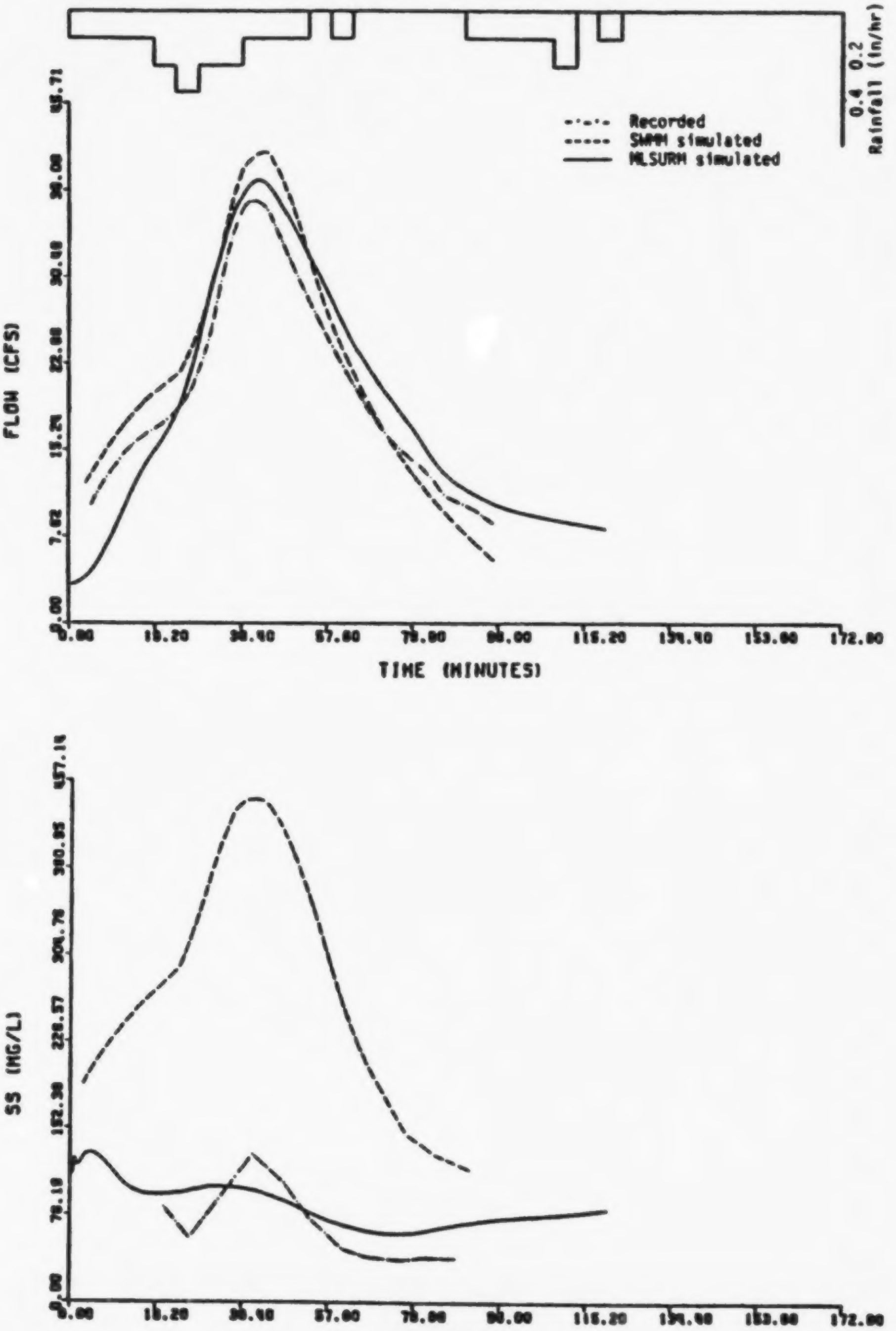


Figure 7 Results from Mortimer Avenue Basin, Toronto, Canada
Storm of June 30, 1976

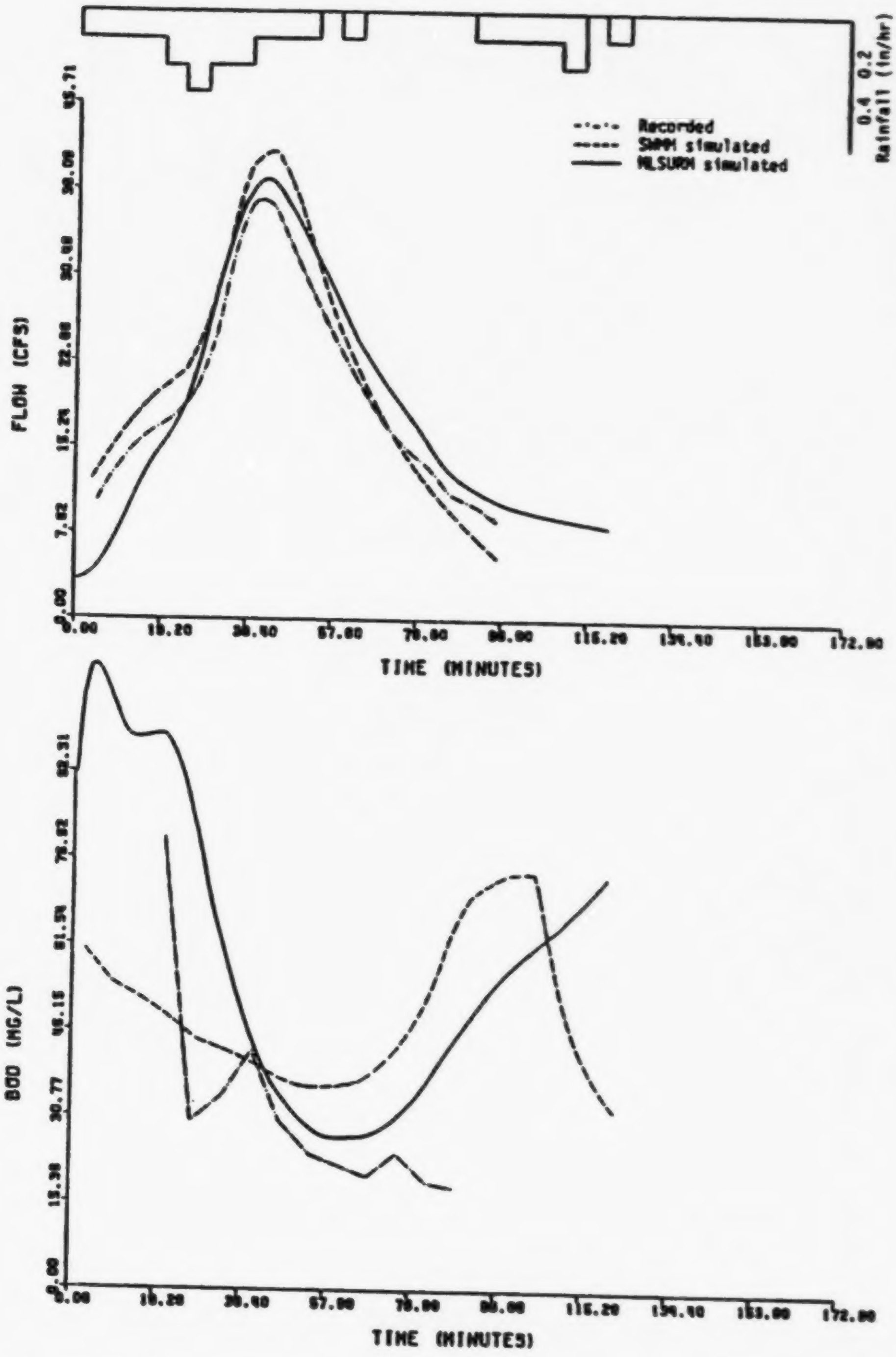


Figure 7 (continued) Results from Mortimer Avenue Basin,
Toronto, Canada
Storm of June 30, 1976

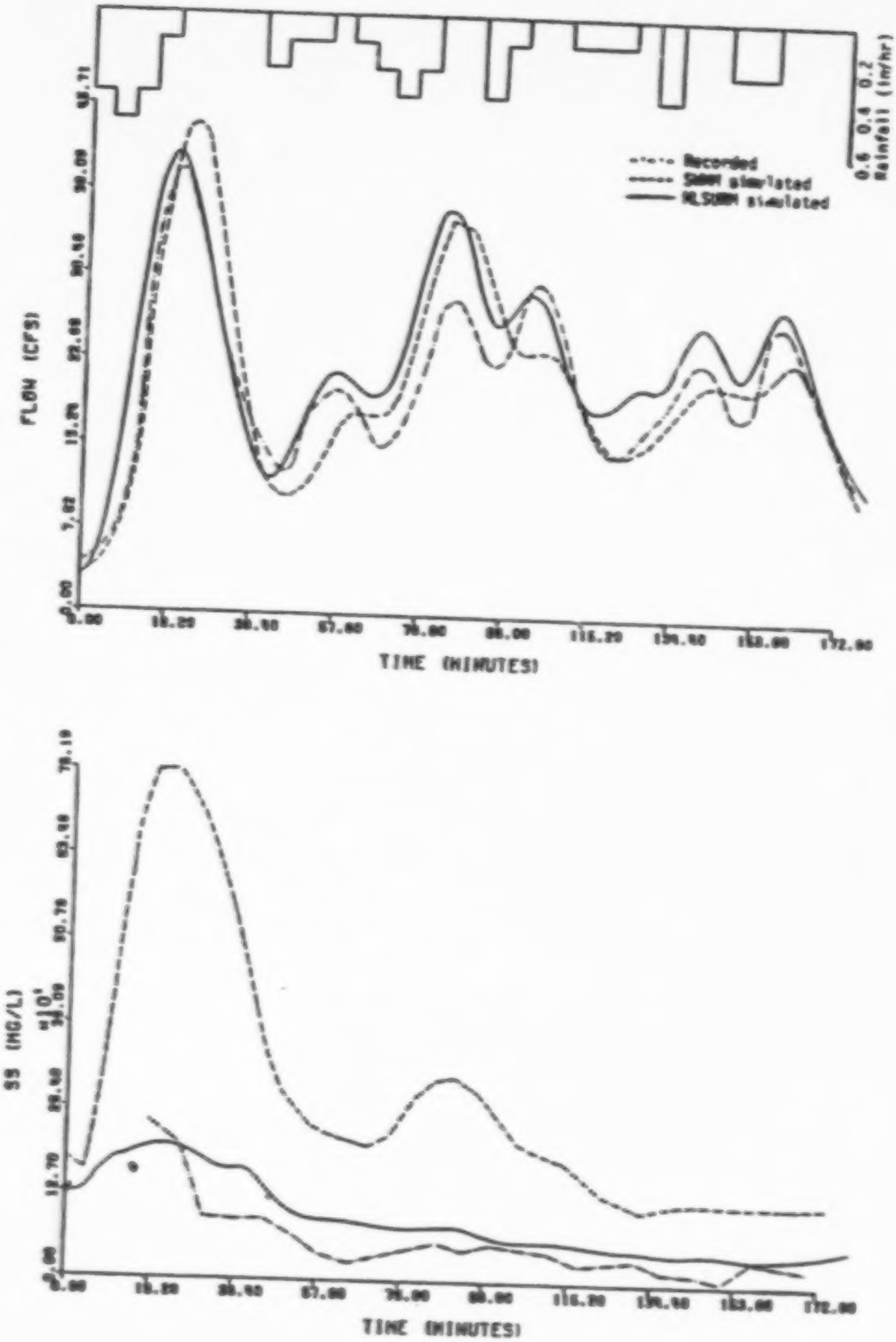


Figure 8 Results from Mortimer Avenue Basin, Toronto, Canada Storm of July 31, 1976

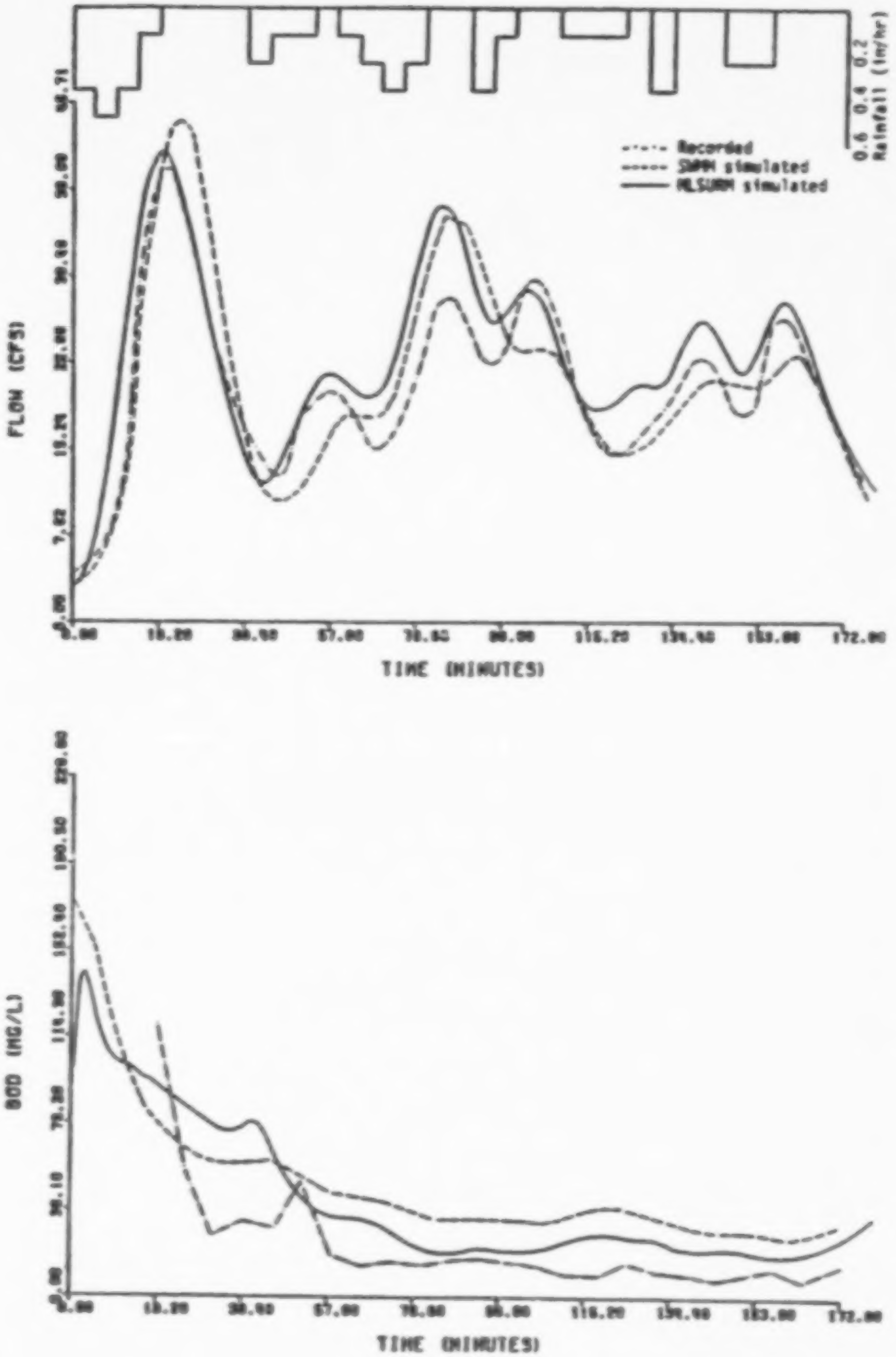


Figure 8 (continued) Results from Mortimer Avenue Basin
Toronto, Canada
Storm of July 31, 1976

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METHODOLOGY FOR 'LUMPED' SWMM MODELLING

by

M. AHMAD[1]

INTRODUCTION

The purpose of this paper is to present a systematic methodology for lumping or aggregating urban drainage areas when using the Storm Water Management Model (SWMM) to simulate the rainfall-runoff process. In a lumped model the study area is discretized into large subcatchments (i.e. coarse discretization) and as such the spatial details of hydrologic characteristics and the internal drainage system of the area are not explicitly modelled. This approach of using coarse discretization in watershed modelling studies considerably reduces the costs of setting up and running the SWMM simulation. However, it also implies that modelling accuracy will suffer unless appropriate steps are taken to account for the effect of the omitted internal details. The main objective of the methodology presented herein is to overcome the above limitation associated with the concept of "lumped modelling".

The growing popularity of SWMM among planners and decision makers over the past few years has expedited the need for developing simplified procedures for lumped simulation. Although use of lumped simulation techniques can considerably reduce the overall modelling costs, selection of a particular discretization level depends entirely on the objective and the nature of the study under consideration. For instance, if detailed information on the hydraulic performance of all major conduits in a basin is required, then it would be necessary to use a detailed discretization scheme.

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Conversely, if the purpose of the study is to develop a hydrograph at the outlet of a catchment, then a coarse discretization through the use of a few larger subcatchments and conduits would be more appropriate.

Edmonton Water and Sanitation (Drainage Engineering Section) has been using SWMM and other models extensively since 1975 for analyzing the existing combined and separated storm trunk systems (Ref. 1). These models are being used mainly for designing relief sewers and for drainage planning purposes. The existing trunk sewer systems in the city serve a total area of over 50,000 acres. By the end of this year, hydraulic analysis of all the major trunk systems would essentially be completed and all data would be stored on the computer for future planning and sewer updating purposes. When analyzing large and complex trunk systems such as those existing in Edmonton, it is essential to employ simplified modelling techniques which reduce the amount of effort required in data preparation and also the simulation costs.

The author, therefore developed the simplified methodology given in this paper for analyzing the existing trunk systems using lumped modelling approach. As a major part of this methodolgy, the concept of equivalent gutter was introduced in RUNOFF block simulation to compensate for the eliminated conduit storage existing within the lumped catchment. In addition, a set of generalized curves relating in-system conduit storage to impervious area were developed using relevant data from new residential and idustrial subdivisions in Edmonton. Similarly, curves relating the drainage area to the peak flow for a range of imperviousness values were also generated. A systematic step-by-step procedure that uses these curves to determine the overland flow "width" parameter and the dimensions of the representative equivalent gutter appropriate for the lumped catchment was formulated.

The lumping methodology presented in this paper was tested against detailed simulations using rainfall and flow measurements for three recorded storm events for the Norwood test area. Modelling results employing lumped and detailed discretization schemes, respectively, were also compared for the 5 - year design storm using the catchment data for Fulton Drive basin. Comparisons of detailed and lumped simulations for both test areas were found to be reasonably good.

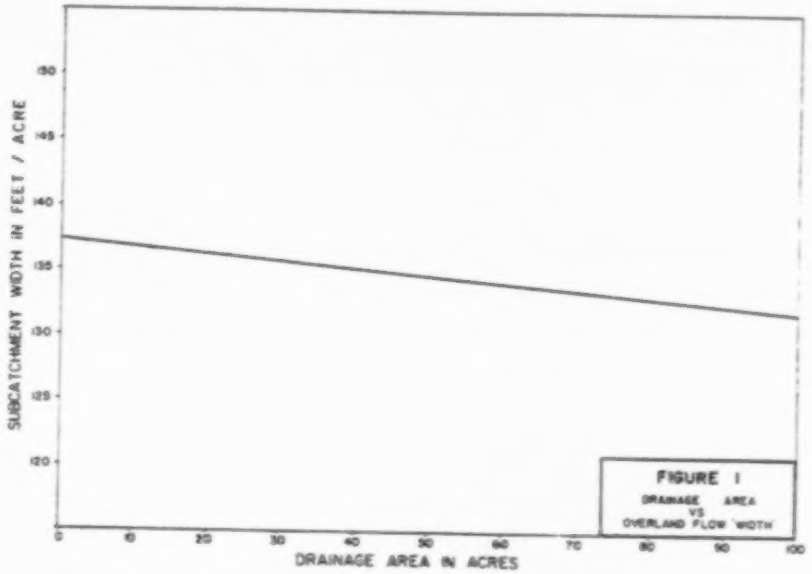
PREVIOUS WORK

The concept of lumping in computer modelling work is not a new one. Literature contains many references on lumped modelling. An excellent reference on this subject is the Canadian Storm Water Management Model Study conducted jointly by J.F. MacLaren Ltd. and Proctor and Redfern Ltd. (Ref. 2). A brief summary of the findings of previous work (Refs. 3,4,5 & 6) related to lumped simulation, is presented in the above Canadian SWMM Report. While the previous studies indicated the feasibility of lumped simulation, they did not evolve and present a systematic procedure or methodology for lumped modelling.

The Canadian SWMM study suggested two alternate methods for introducing additional storage into the lumped model so as to essentially compensate for the unmodelled conduit storage. The study showed that the designed attenuation and the time delay of the hydrograph, which are critical for the accuracy of the lumped simulation, could be achieved either by increasing the length of the aggregated conduit or by reducing the overland flow "width" of the "lumped" catchment. The study concluded that it was possible to conduct an accurate lumped simulation for a particular catchment by using average values for parameters describing surface runoff characteristics of the lumped catchment (ie. infiltration, detention depth, ground slope, Manning's 'n'). However, the study also indicated that it was not possible to determine the appropriate value of overland flow "width" for the lumped catchment without first conducting a detailed simulation. Similarly, the length of the equivalent gutter needed to produce consistent results from lumped simulation could not be determined without conducting a detailed simulation. As a result, the study was unable to recommend a generalized procedure for the lumped simulation.

METHODOLOGY FOR LUMPED SIMULATION

Previous work has shown that an accurate lumped simulation can be carried out either by reducing the value of overland flow "width" parameter in RUNOFF block or by selecting a "reasonably" large equivalent conduit to compensate for the omitted conduit storage in the lumped catchment (Ref. 2). Results of a previously conducted RUNOFF block sensitivity analysis study (Ref. 7) indicated that while it was difficult to relate the value of "width" parameter to the eliminated in-system conudit storage, it was much easier to determine the size and length of the equivalent gutter for a lumped catchment. Therefore, a simplified procedure was formulated to estimate the section dimensions and the length of the equivalent gutter for general use in a lumped simulation. Based on the results of our sensitivity analysis, a relationship between catchment "width" and drainage area was derived as illustrated in Figure 1. "Width" values in feet/acre appear not to be very sensitive to changes in the drainage area size. Further, it was discovered that the peak flow was not sensitive to the changes in "width" especially at the higher "width" values as shown in Figure 2. The total runoff volume was also found to be quite insensitive to the variation in "width" as shown in Figure 2. Using the results of



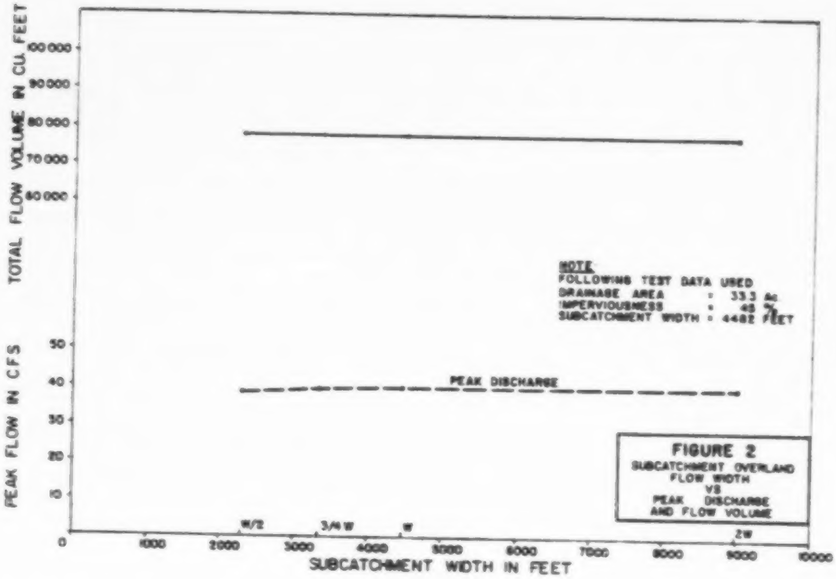


FIGURE 2
SUBCATCHMENT OVERLAND FLOW WIDTH VS PEAK DISCHARGE AND FLOW VOLUME

NOTE
FOLLOWING TEST DATA USED
DRAINAGE AREA : 33.3 Ac
IMPERVIOUSNESS : 45 %
SUBCATCHMENT WIDTH : 4482 FEET

this sensitivity analysis, a simplified lumping procedure described below was developed. This step-by-step procedure can be used to determine the necessary input parameters for a lumped catchment. For simplicity, a rectangular equivalent gutter is suggested for lumped simulation.

Lumping Procedure

1. Determine the average values of various parameters describing surface runoff characteristics including imperviousness ratio, "width", infiltration, detention depth, ground slope and Manning's 'n'. (value of "width" can be estimated using the curve shown in Figure 1).

2. Calculate total length of each sewer size in feet.

3. Calculate available conduit storage in cubic feet for each sewer size by multiplying the cross-sectional area of the sewer and the total length computed above in step 2.

4. Calculate total available storage by adding storage values for all sewer sizes.

5. Estimate the 5-year design peak inflow (or any other flow as required) to the equivalent gutter. For the City of Edmonton, the 5-year design storm flows can be estimated from the generalized curves shown in Figure 3. These curves were developed using the results of RUNOFF block simulations for a number of newly developed residential and industrial areas. These curves show a relationship between drainage area and the peak flow for 3 different imperviouness ranges for the 5-year design storm. The peak flow values used for these curves are pre-routing flows. For example, curve 1 (Figure 3) represents a relationship between drainage area (acres) and peak discharge (cfs) for catchments with imperviouness ratio ranging from 15% to 30%.

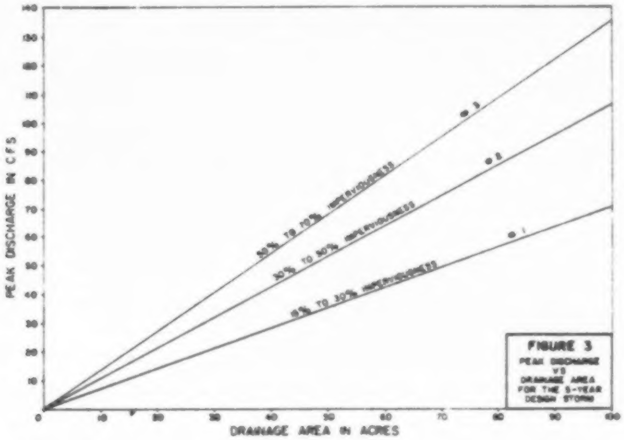
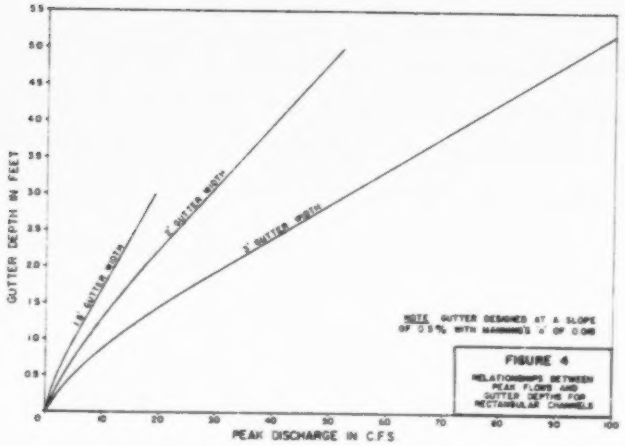


FIGURE 3
PEAK DISCHARGE
VS
DRAINAGE AREA
FOR THE 5-YEAR
DESIGN STORM

6. Select an appropriate width 'b' for the equivalent gutter (rectangular) based on the peak inflow estimated in step 5. By using Figure 4 (which shows relationships between peak inflow and gutter depth for a number of selected widths), determine depth of flow 'd' in the gutter for the peak inflow obtained in step 5. (Equivalent gutter is designed at a slope of 0.5% with a Manning's 'n' value of 0.018.)

7. Determine the length of the equivalent gutter in feet by dividing the total storage volume computed in step 4 by the cross-sectional area of the gutter (b x d).



NOTE: GUTTER DESIGNED AT A SLOPE OF 0.5% WITH MANNING'S 'n' OF [illegible]

FIGURE 4
RELATIONSHIPS BETWEEN PEAK FLOWS AND GUTTER DEPTHS FOR RECTANGULAR CHANNELS

It is evident from the above procedure that in order to estimate in-system conduit storage accurately, one has to spend a considerable amount of time, especially if a large basin is involved. Also, the above method cannot be applied directly to the presently undeveloped areas. Therefore, to overcome these problems, two generalized curves relating the in-system conduit storage to the impervious drainage areas for predominantly residential and industrial areas, respectivley, were developed using regression analysis techniques. Sewer system data for the newly developed residential and industrial subdivisions were used for the statistical analysis. Figure 5 shows the curves for both residential and industrial areas. Storm sewer system in these areas have been designed to carry the 5-year design storm flows. The total amount of in-system conduit storage can be estimated using curves shown in Figure 5 instead of going through

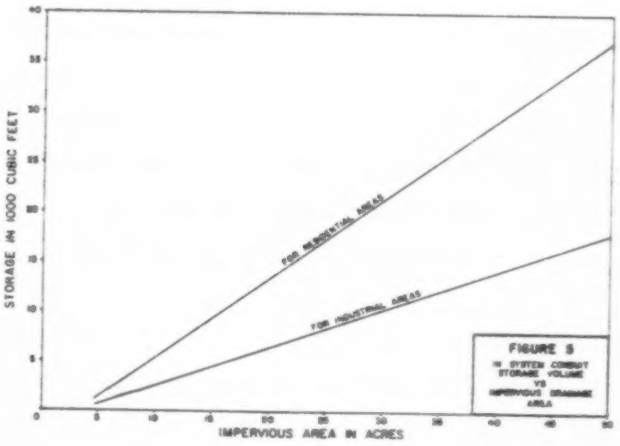


FIGURE 5
IN SYSTEM CONDUIT STORAGE VOLUME VS IMPERVIOUS DRAINAGE AREA

steps 2 to 4 as suggested in the lumping procedure. Again, curves shown in Figures 3 and 5 are valid only for Edmonton conditions and their use for other cities is not recommended.

TESTING AND APPLICATION OF LUMPING METHODOLGY

Fulton Drive Test Catchment

The proposed lumping methodology was tested using Fulton Drive test catchment data for the 5-year design storm conditions. A 38.91 acre subarea with an average imperviousness of 34.12% was selected for comparison purposes. Schematics of the test area and the sewer system are shown in Figures 6 and 7, respectively. The test area was divided into 12 subcatchments for a detailed simulation and input data was prepared using the procedure given in the SWMM User's Manual (Ref. 3). Equivalent parameters were then estimated using the lumping procedure described earlier to allow the entire 38.91 acre catchment to be modelled as a

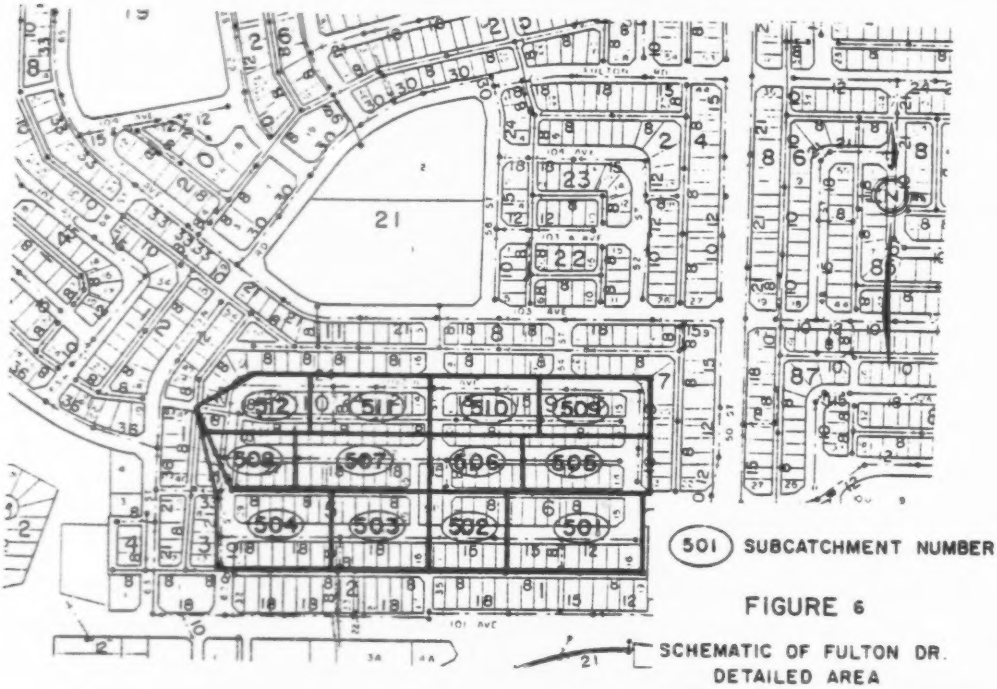


FIGURE 6

SCHEMATIC OF FULTON DR.
DETAILED AREA
SUBCATCHMENT BOUNDARIES

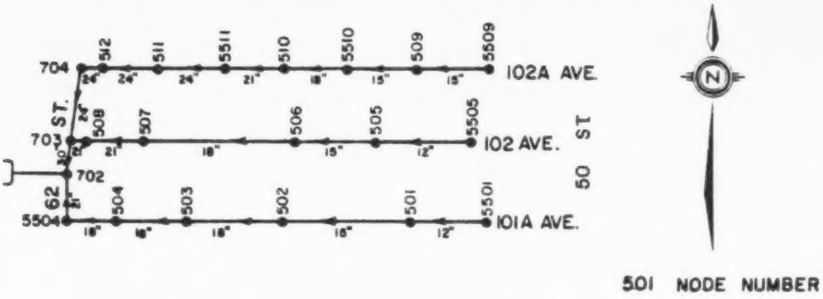

704
512
511
5511
510
5510
509
5509
102A AVE.
24"
24"
24"
21"
18"
15"
15"
62 ST.
24"
703
508
507
506
505
5505
102 AVE.
30"
21"
21"
18"
15"
12"
50 ST
702
21"
5504
504
503
502
501
5501
101A AVE.
18"
18"
18"
18"
12"
N
501 NODE NUMBER

FIGURE 7

SCHEMATIC OF FULTON DR.
DETAILED AREA SEWER SYSTEM
AS MODELLED

single catchment. Equivalent parameters were also estimated by dividing the entire area into 3 subcatchments. SWMM simulations were carried out using the 5-year design storm for three cases where the study area was represented by a single lumped catchment, 3 subcatchments and 12 subcatchments respectively. For detailed simulation with 12 subcatchments, all sewers were modelled in the TRANSPORT block. Lumped simulations with one catchment and with 3 subcatchments were carried out using only the RUNOFF block.

The results of the simulations are summarized in Table 1 and the simulated hydrographs are shown in Figure 8. Detailed and lumped simulations compare reasonably well for the tested conditions. However, it should be noted that the simulated peak flow rate with single lumped catchment was about 20% higher than that with detailed discretization using 12 subcatchments, when both simulations were made with a 5-year design storm (Chicago type) discretized into average 5 minute intensities. As illustrated in Figure 8, improved simulation results were obtained for the lumped simulation when the same design storm was discretized by averaging the highest peak intensities over 15 minutes (which happens to be the time of concentration of the catchment). Difference in simulation results with 3 subcatchments and 12 subcatchments was insignificant. Therefore, it can be concluded that as part of the above lumping procedure, the design storm

TABLE 1

FULTON DRIVE TEST CATCHMENT
LUMPED SIMULATION
(5-YEAR DESIGN STORM)

Number of Subcatchments	Hyetograph Discretization Interval (minutes)	Overland Flow Width (Feet)	Peak Flow (cfs)	Total Runoff Volume (cu. ft.)
1	5	4,900	37.5	69,162
1	15	4,900	32.0	69,162
3	5	4,900	33.0	68,946
12	5	9,800	31.2	68,958

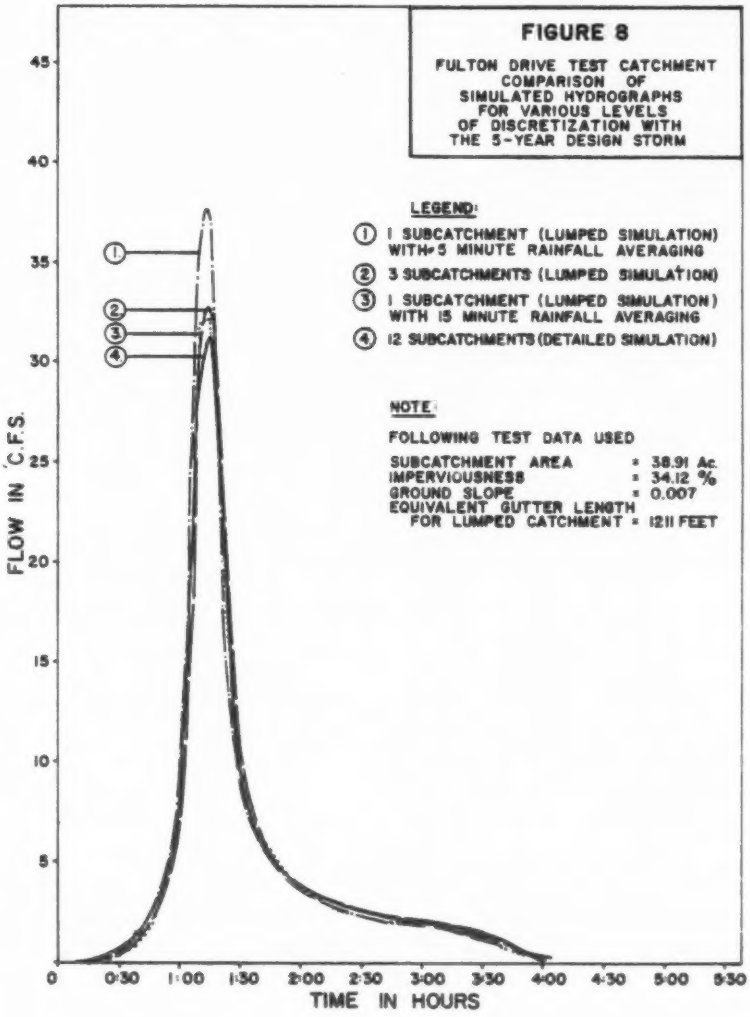


FIGURE 8

FULTON DRIVE TEST CATCHMENT COMPARISON OF SIMULATED HYDROGRAPHS FOR VARIOUS LEVELS OF DISCRETIZATION WITH THE 5-YEAR DESIGN STORM should be discretized by averaging the highest intensities occurring during an interval of time approximating the estimated time of concentration of the catchment under consideration.

Norwood Test Area

The lumping mthodology was also tested using data for the Norwood area. This is an old residential neighbourhood in which relief sewers were constructed in 1976-77. A 32.6 acre subcatchment in this area was selected for the lumped simulation testing purposes. Imperviousness of the area is approximately 26%. A schematic of the test area is shown in Figure 9 and the existing storm sewer sytem is illustrated in Figure 10. The test area is drained by a 36" storm sewer which discharges into an 111" storm trunk along 111th Avenue.

Detailed SWMM simulations were carried out by discretizing the test area into 19 subcatchments ranging in size from 1 acre to 3 acres and by using

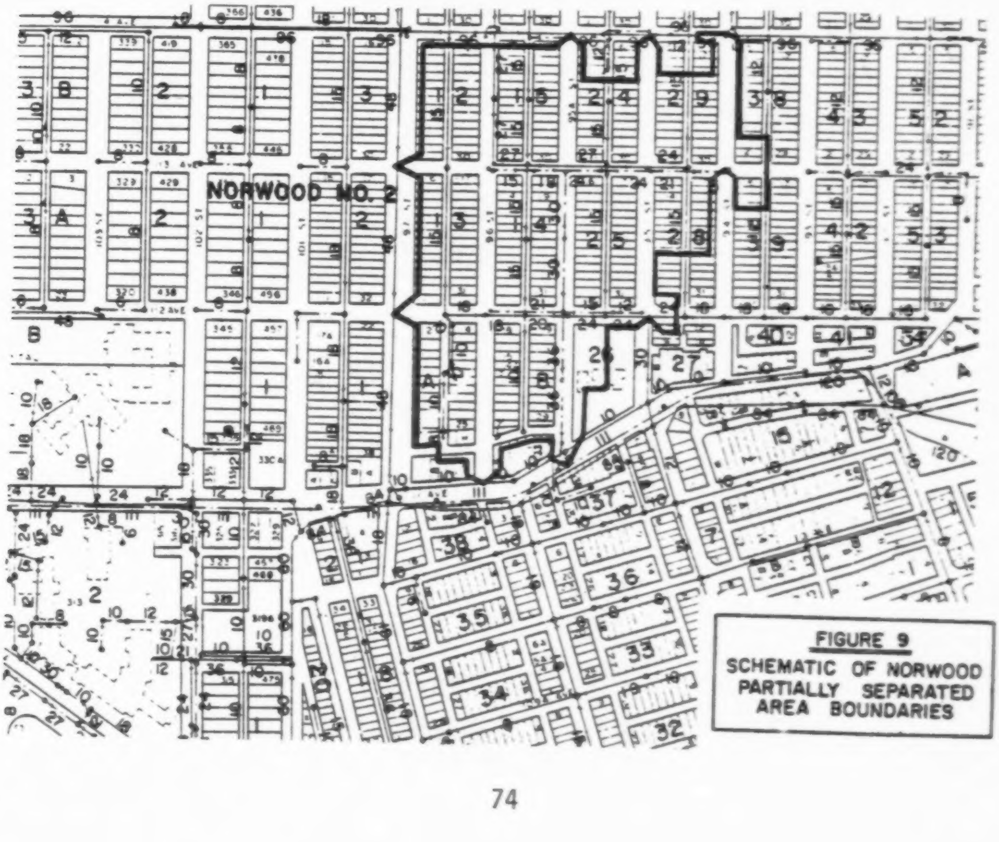


FIGURE 9
SCHEMATIC OF NORWOOD PARTIALLY SEPARATED AREA BOUNDARIES

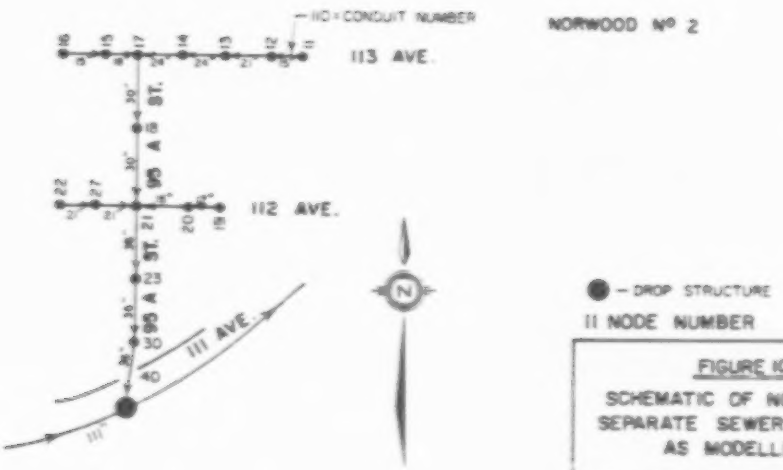
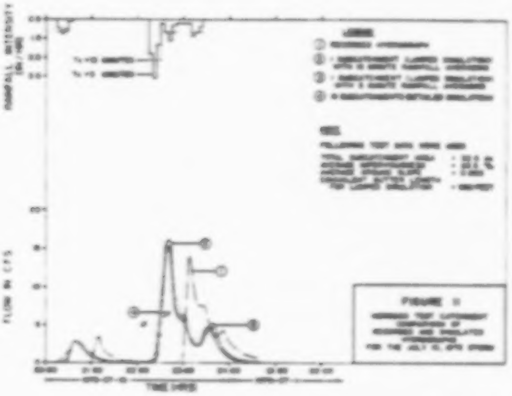
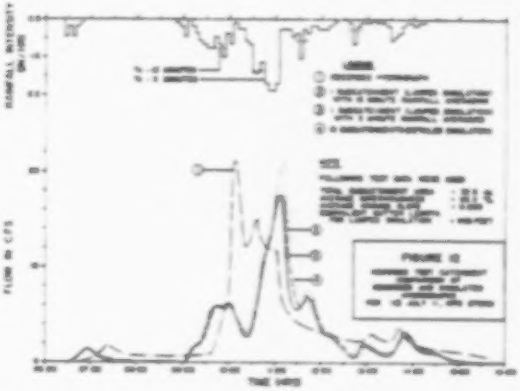


FIGURE 10
SCHEMATIC OF NORWOOD
SEPARATE SEWER SYSTEM
AS MODELLED

input data prepration procedures as outlined in the SWMM User's Manual (Ref. 3). The input parameters for lumped SWMM simulations were estimated using the lumping methodology presented in this paper. Rainfall and flow data for the three recorded storm events (July 10th and 11th, 1978 and June 14th, 1979) were used to compare the recorded hydrographs with those computed by SWMM for detailed and lumped simulations, respectively. Lumped simulations were carried out using both 5 and 15 minutes hyetograph discretization levels as described earlier for the Fulton Drive area. The recorded hydrographs are compared with lumped and detailed simulation hydrographs in Figures 11, 12, and 13. A very good comparison is found with 15 minute hyetograph descritization for the tested storm events. The results of these simulations are summarized in Table 2. These results clearly indicate that by properly applying the lumping methodology, it is possible to achieve an accurate lumped simulation. Furthermore, it confirms that input rainfall hyetograph should always be discretized according to the catchment size. For instance, if the estimated time of concentration of an area is 20 minutes, then the input rainfall hyetograph should be discretized by averaging the highest rainfall intensities over a time interval of 20 minutes.

This lumping methodology which is also summarized in Figure 14 has been applied successfully in the City of Edmonton for analyzing the existing





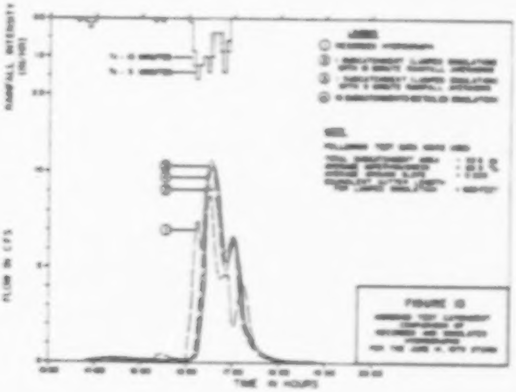


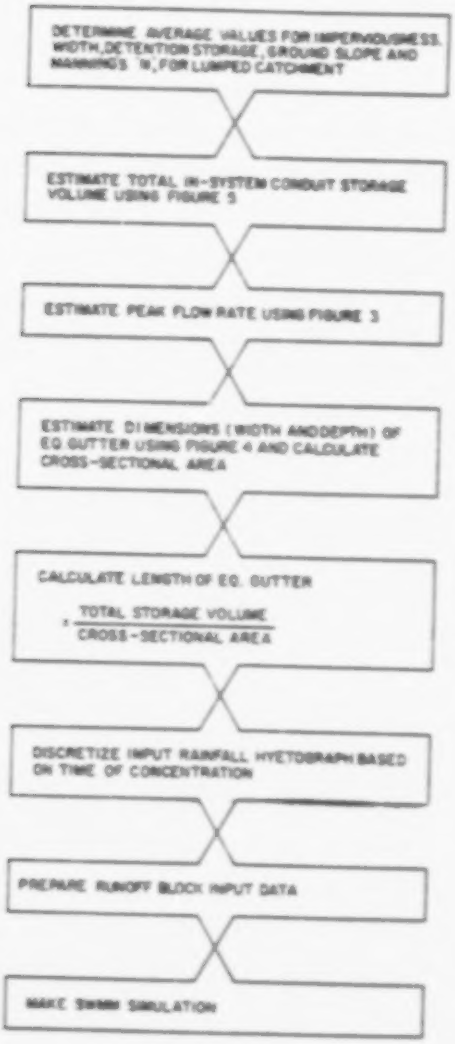
TABLE 2

NORWOOD TEST AREA
LUMPED SIMULATION
(RECORDED STORMS)

Storm	Number of Subcatchments	Hyetograph Discretization Interval (Minutes)	Overland Flow Width (Feet)	Peak Flow (cfs) Rec.	Peak Flow (cfs) Sim.	Total Simulated Runoff Volume (cu. ft.)
1978 07 10	19	5	14,433	14.0	15.3	33,500
1978 07 10	1	15	4,458	14.0	15.4	35,100
1978 07 10	1	5	4,458	14.0	16.2	35,100
1978 07 11	19	5	14,433	21.2	21.2	92,500
1978 07 11	1	15	4,458	21.2	17.6	88,500
1978 07 11	1	5	4,458	21.2	17.5	88,500
1979 06 14	19	5	14,433	8.8	10.3	26,500
1979 06 14	1	15	4,458	8.8	9.5	26,900
1979 06 14	1	5	4,458	8.8	10.6	25,900

FIGURE 14

METHODOLOGY FOR LUMPED
SWMM RUNOFF SIMULATION
FOR URBAN AREAS


DETERMINE AVERAGE VALUES FOR IMPERVIOUSNESS, WIDTH, DETENTION STORAGE, GROUND SLOPE AND MANNINGS 'N', FOR LUMPED CATCHMENT
ESTIMATE TOTAL IN-SYSTEM CONDUIT STORAGE VOLUME USING FIGURE 5
ESTIMATE PEAK FLOW RATE USING FIGURE 3
ESTIMATE DIMENSIONS (WIDTH AND DEPTH) OF EQ. GUTTER USING FIGURE 4 AND CALCULATE CROSS-SECTIONAL AREA
CALCULATE LENGTH OF EQ. GUTTER
= TOTAL STORAGE VOLUME / CROSS-SECTIONAL AREA
DISCRETIZE INPUT RAINFALL HYETOGRAPH BASED ON TIME OF CONCENTRATION
PREPARE RUNOFF BLOCK INPUT DATA
MAKE SWMM SIMULATION

storm trunk systems as well as for designing new storm trunks. At present, studies are underway to develop similar procedures for larger developing drainage areas.

CONCLUSIONS AND RECOMMENDATIONS

The simplified lumping procedure described in this paper can be applied for analysing and designing of storm sewer systems in urban basins without using detailed SWMM simulations. A significant reduction both in the amount of effort required in input data prepration and in the overall simulation costs can be achieved by employing this lumping methodology. Generalized curves relating impervious drainage area and in-system conduit storage similar to those given in this paper for Edmonton can be developed for other cities. Further studies should be undertaken to develop simplified methods for other SWMM applications such as storage volume estimation for stormwater lakes, runoff peak and volume computations for pre-development conditions, etc.

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CHARACTERIZATION, MAGNITUDE AND IMPACT OF URBAN RUNOFF IN THE GRAND RIVER BASIN

by

S.N. SINGER and S.K. SO
Water Resources Branch
Ministry of the Environment

INTRODUCTION

The general goals of the Grand River Basin Water Management Study are:

1- to develop viable water management options needed to plan for, and encourage, the integrated use of water and land resources, within the Grand River Basin.

2- to identify the necessary trade-offs to achieve protection against flooding, acceptable disposal and transport of waste effluents.

3- to provide adequate supplies of good quality water to meet water supply, aesthetic, fish, wildlife and recreation desires and needs.

4- to ensure a productive and fulfilling environment for the people of the basin.

In summary, the three water management objectives of the Grand River Basin Study are to reduce flood damages, to provide adequate water supply and to maintain an acceptable water quality.

Water quality constitutes an important component of the study. The key elements of water quality investigation are:

1- to determine existing water quality conditions and relate them to various water uses.

2- to identify the type, magnitude, relative significance and impact of pollutants from point and nonpoint (urban and rural) sources on the water quality of the river.

3- to develop water quality management programs to preserve areas of high water quality and to upgrade areas where poorer water quality exists.

Urban stormwater runoff has been recognized as a potential major contributor of pollution to the Grand River. Therefore, investigation of pollution from urban sources became an integral part of the basin's water quality assessment program. Because costs associated with the abatement of urban stormwater pollution range in the tens of millions of dollars it was important to assess the impact of this pollution on the river and to determine its significance.

This paper contains a description of the Grand River Basin and its major urban centres and an overview of urban data collection programs. Significant results to date related to the characterization of urban runoff, magnitude of pollution loads and their impact on the Grand River are presented.

DESCRIPTION OF THE GRAND RIVER BASIN

The Grand River Basin is located in southwestern Ontario between longitudes 79^0 30' and 80^0 57' W, and latitudes 42^0 51' and 44^0 13' N. The basin occupies the central part of a peninsula bounded on the north by Georgian Bay, on the west by Lake Huron, on the south by Lake Erie and on the east by Lake Ontario (Figure 1). The basin has an area of about 6,700

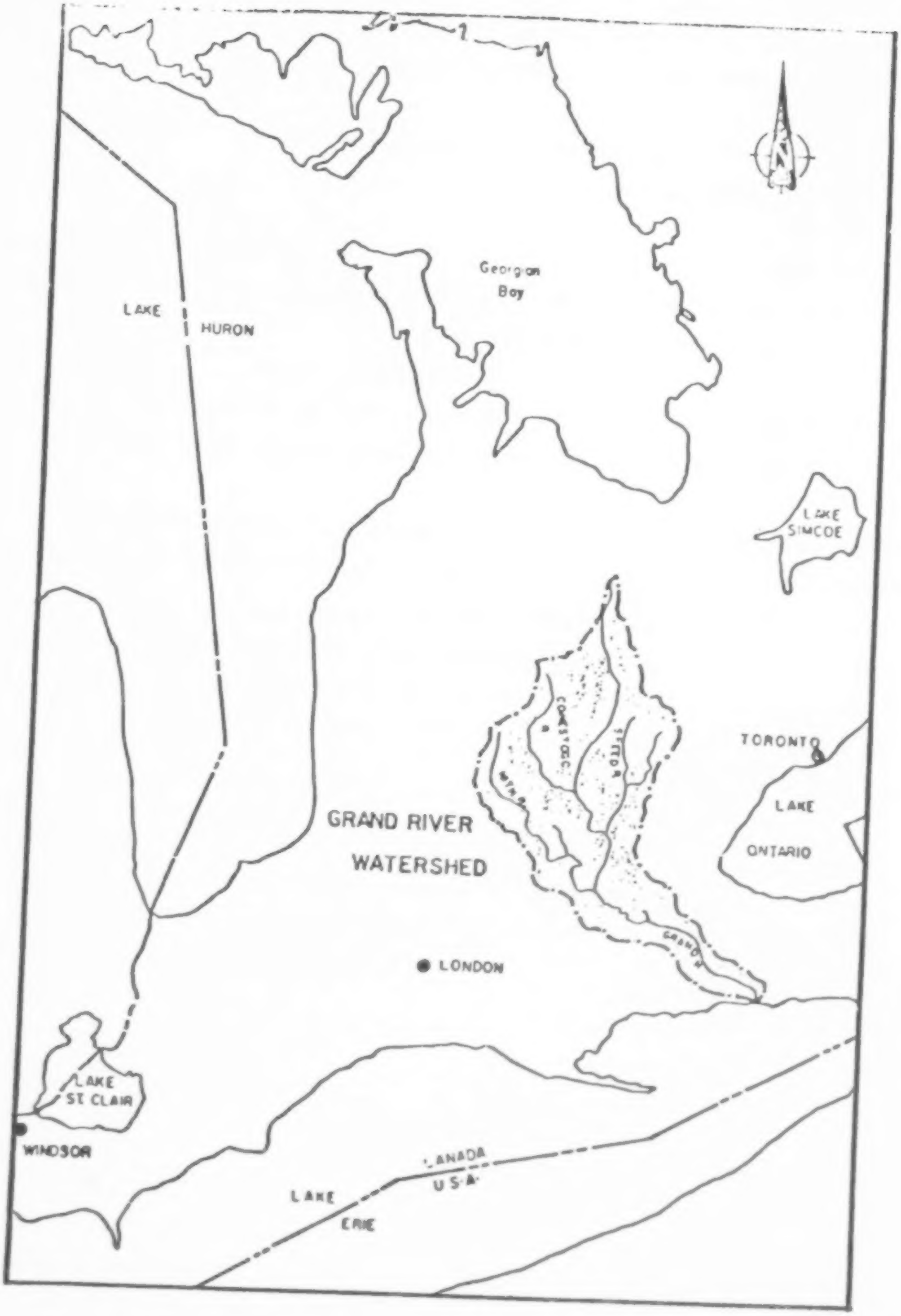
Georgian
Bay
LAKE HURON
LAKE SIMCOE
TORONTO
LAKE ONTARIO
GRAND RIVER WATERSHED
CONESTOGO
SPEED R
NITH R
GRAND R
LONDON
LAKE ST CLAIR
WINDSOR
CANADA
U.S.A.
LAKE ERIE

Figure 1. Location of the Grand River Basin in Southwestern Ontario.

km^2, a length of about 290 km and a width which varies between 5 and 75 km.

The headwaters of the Grand River rise in a massive swampy upland south of Georgian Bay at an elevation of approximately 526 m above mean sea level (msl). The river flows in a southerly direction until it reaches the Town of Paris. From there it follows a southeasterly direction to discharge into Lake Erie at Port Maitland at an elevation of 174 m above msl. The Conestogo, Speed and Nith rivers are the three major tributaries which join the main stem in the middle portion of the basin. The Conestogo River drains the northwestern portion of the basin with the Speed and Nith rivers draining the eastern and western portions of the basin, respectively. In the upper part of the basin, the Grand and its tributaries flow in previously formed glacial spillway channels. In the lower part, below the City of Brantford, the river has scoured its own channel across glacial lake deposits of silt and clay.

The drainage basin is characterized by a temperate climate that receives a moderating influence from the nearby Great Lakes System. The long-term mean annual temperatures vary from 6°C in the headwaters to 9°C at Lake Erie. The long-term mean annual precipitation varies from 84 cm (178 cm snow) in the lower reaches to 88 cm (127 cm snow) in the upper reaches of the basin.

The mean annual flow at the outlet of the river is estimated to be 64 m^3/s which corresponds to a mean annual runoff of 30 cm of precipitation. Peak flows range from 500 to 1400 m^3/s. In general, peak flows occur during the spring melt period. The highest flow on record, however, occurred as a result of Hurricane Hazel in November of 1954.

LAND USE, POPULATION AND MAJOR URBAN CENTRES

The Grand River Basin has been developed extensively for urban and agricultural uses which comprise 3% and 75% of the total basin area, respectively. Wooded and/or idle areas account for approximately 19% of the basin area and the remaining 3% lies in other uses. Urban uses are predominant in the central portion of the basin where the cities of Kitchener, Waterloo, Guelph, Cambridge and Brantford are located (Figure 2).

The population of the basin is approximately 545,000 and is primarily concentrated in the above mentioned five large urban centres. This, however, was not always the case. In 1921, 43% of the basin's population was rural and only 47.5% lived in the five urban centres (Table 1). By 1976, however, the proportion of rural population dropped to about 20% whereas the proportion of population in the five urban centres increased to 72%. If these trends continue through subsequent years, it can be expected that the urban population by the year 2001 might increase by about 200,000 to 300,000 people. This could increase pollution loadings to streams from urban runoff, further impair water quality conditions and tax available water supplies.

At present, water quality problems in the upper or northern third of the basin, where agriculture is predominant, are related primarily to erosion. Soils and nutrients are carried into the river causing turbidity, nutrient enrichment and limited localized algal growth. The algal density (bottom cover) varies from 0-80% coverage of the streambed (Figure 3). With the exception of localized aquatic growth, water quality problems in this part of the basin are usually limited in extent and do not have significant detrimental impact on river uses.

The heavily urbanized and industrialized central third of the basin is significantly affected by the discharge of treated domestic and industrial wastes and urban stormwater runoff. Two of the major problems that have been identified through the Grand River Study are: 1) the reduction of dissolved oxygen in areas downstream from the major municipalities caused by municipal discharges of oxygen-consuming wastes; and 2) profuse algae and plant growths stimulated by nutrient inputs from point and nonpoint sources.

TABLE 1

POPULATION IN THE GRAND RIVER BASIN

Cities	1921		1941		1966		1976		Population Projections for 2001**		
									Low		Medium
Kitchener	21,763		35,657		91,376		131,801		200,795		227,486
Waterloo	5,883		9,025		29,770		49,972		86,461		92,131
Cambridge	21,416		25,108		51,482		71,482		46,815		124,474
Guelph	18,128		23,273		49,497		70,374		90,250		115,456
Brantford	29,440		31,948		58,395		66,930		85,833		94,982
Total	96,630		125,011		280,520		390,599		588,159		649,529
% of watershed population	47.5		53.0		68.2		71.6		[illegible]		N/A
Average Annual Growth Rate %		1.3		3.3		3.4		1.8*		2.3*	
Incorporated Towns and Villages											
Total Population	18,589		20,818		35,961		43,559		58,896		69,114
% of watershed population	9.1		8.8		8.7		8.0		7.3		N/A
Average Annual Growth Rate %		.6		2.2		2.1		1.57*		2.27*	
Rural Areas (including unincorporated rural hamlets)											
Total population	88,204		89,795		95,118		111,185		200,947		N/A
% of watershed population	43.4		38.0		23.1		20.4		22.9		N/A
Average Annual Growth Rate %		.09		.23		1.5		1.81*			
Total watershed population	203,423		235,624		411,599		535,051		877,137		N/A
Average Annual Growth Rate %		.74		2.3		2.8		1.92*			

* Growth rates apply for the years between 1976 and 2001.
** Population projections based upon available data, March 7-79.

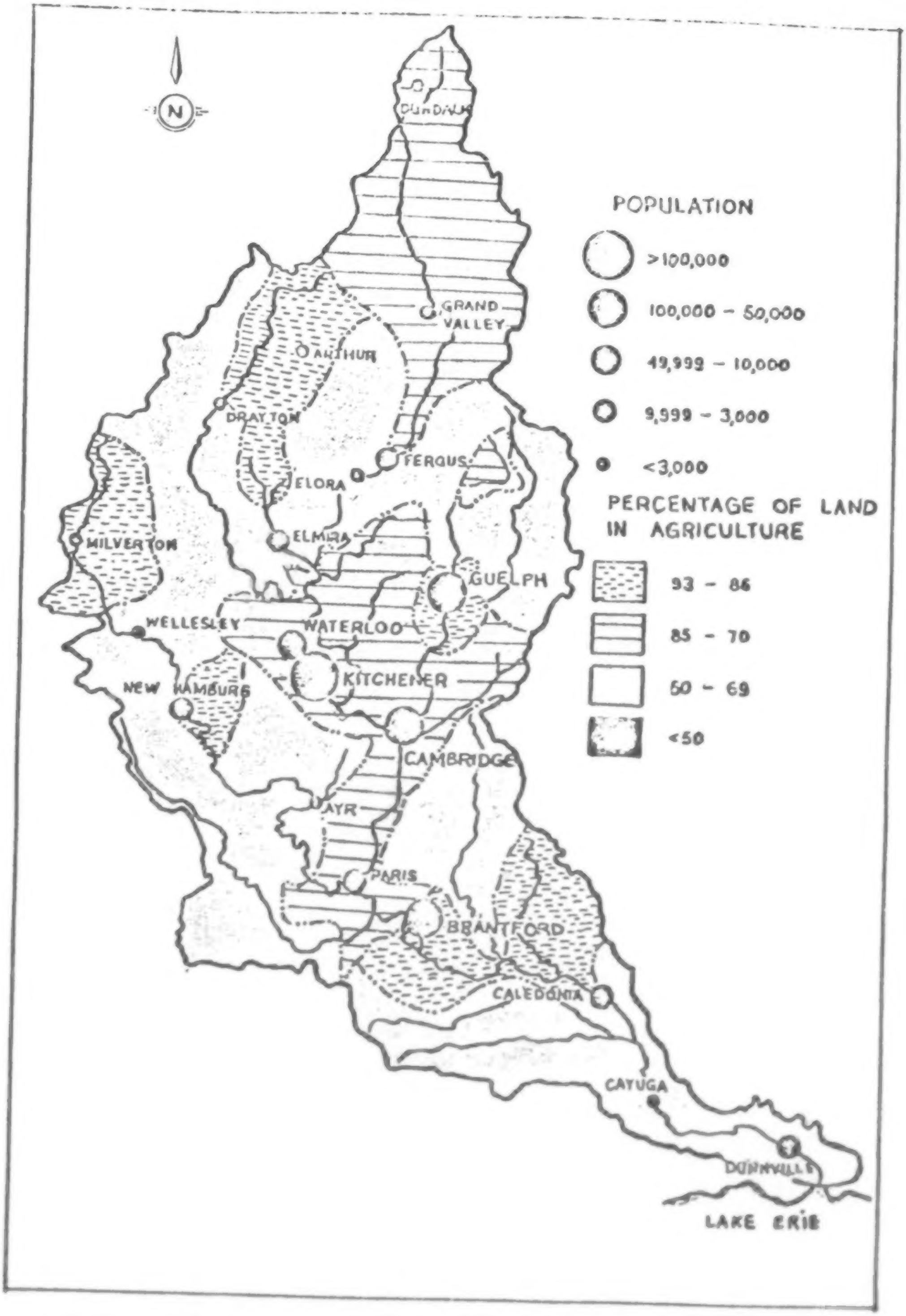
N
DUNDALK
POPULATION
>100,000
100,000 - 50,000
49,999 - 10,000
9,999 - 3,000
<3,000
PERCENTAGE OF LAND IN AGRICULTURE
93 - 86
85 - 70
50 - 69
<50
GRAND VALLEY
ARTHUR
DRAYTON
FERGUS
ELORA
ELMIRA
MILVERTON
GUELPH
WELLESLEY
WATERLOO
KITCHENER
NEW HAMBURG
CAMBRIDGE
AYR
PARIS
BRANTFORD
CALEDONIA
CAYUGA
DUNNVILLE
LAKE ERIE

Figure 2: Population and Land Use Distribution in the Grand River Basin.

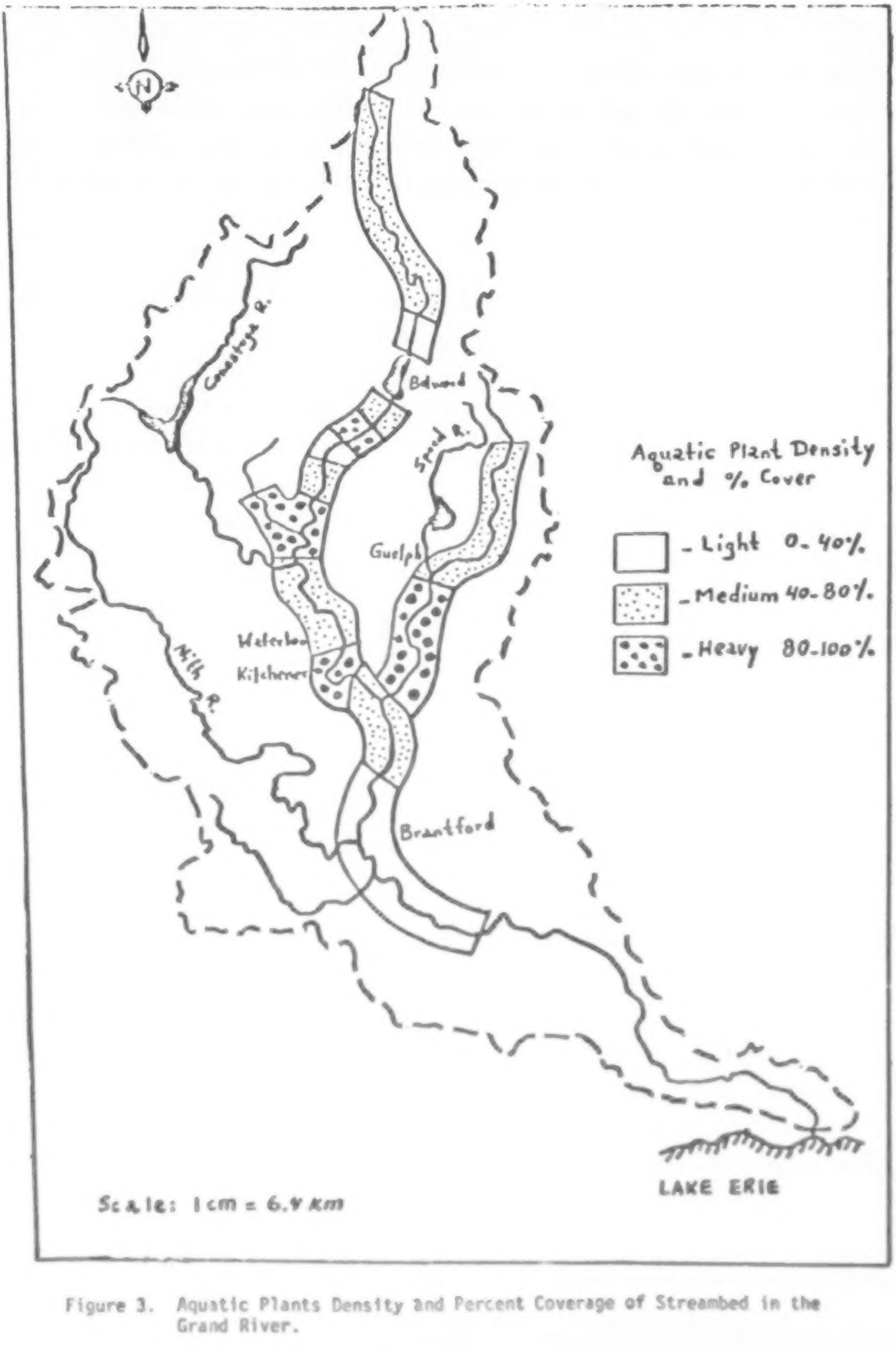
N
Conestogo R.
Belwood
Speed R.
Guelph
Waterloo
Kitchener
Nith R.
Brantford
Aquatic Plant Density and % Cover
- Light 0-40%
- Medium 40-80%
- Heavy 80-100%
LAKE ERIE
Scale: 1cm = 6.4 km

Figure 3. Aquatic Plants Density and Percent Coverage of Streambed in the Grand River.

The lands of the lower third of the basin and the Nith River, a major tributary draining the western portion of the watershed, are primarily used for agriculture and the problems encountered there are quite similar to those identified earlier for the northern third of the watershed (Jeffs et al, 1978).

The land use distribution and drainage systems in the five major cities in the basin are described in the following sections.

Brantford - The City of Brantford is located on the Grand River approximately 91 km north of Lake Erie. The City has a developed area of 3214 ha. and it is served by separate sanitary and storm sewer systems. Approximately 30% of the city area is drained via the storm sewer system directly to the Grand River, 35% to Fairchild's Creek, 30% to Mohawk Creek, 3% to Paper Mill Creek and 2% to D'Aubigny Creek. The land uses are: residential 52.6%, commercial 4.9%, institutional 4.5%, industrial 16.0% and open space* 21.7%.

Brantford is the centre of agricultural machinery manufacturing in Canada and it produces a wide range of goods from chemicals, building products, pulp products, refrigeration units and construction machinery to school buses and transport trailers.

Cambridge - The City of Cambridge was created in 1973 as a result of the amalgamation of the towns of Hespeler, Preston and Galt. Hespeler lies 7.3 km upstream on the Speed River while Preston is located at the confluence of the Speed and Grand Rivers and Galt lies 6.3 km downstream on the Grand River. Cambridge has a total developed area of 3675 ha. The land uses are: residential 52.5%, commercial 10.3%, institutional 4.5%, industrial 14.0% and open space 18.7%.

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* parks, cemeteries ... etc.

Each of the three amalgamated towns has its own separate storm sewer system. The storm sewer system in Hespeler discharges to the Speed River whereas in Preston it discharges to both the Speed River (50%) and the Grand River (50%). Approximately 30% of storm runoff from Galt discharges to Galt Creek and the remaining 70% to the Grand River.

Guelph - Guelph is located at the confluence of the Speed and Eramosa Rivers. The city has a developed area of 3080 ha which has been divided into the following land uses: residential 50.5%, commercial 3.5%, institutional 12.8%, industrial 13.6% and open space 19.6%.

The City has completely separate sanitary and storm sewer systems. The storm sewer system discharges into the Speed River (70%) and Eramosa River (30%).

Modern stormwater management concepts are being implemented in some of the developing subdivisions in the city. Ponds are used in three new residential subdivisions for sedimentation and ground water recharge; another pond is under construction in a developing industrial zone.

Kitchener - Waterloo - The twin cities of Kitchener and Waterloo are located in the central part of the Grand River Basin. The cities have a combined developed area of 7864 ha which have the following land uses: residential 39.7%, commercial 5.9%, institutional 2.9%, industrial 14.7% and open space 36.8%.

The City of Waterloo is served by separate sanitary and storm sewer systems. The storm sewer system discharges at several locations along Laurel Creek and its tributaries. The City of Kitchener has essentially separate sanitary and storm sewer systems except in the old section of the City where house foundation drains are connected to the sanitary sewers because the storm sewers are placed at inadequate depths. Schneider Creek and its tributary, Montgomery Creek, receive about 80% of Kitchener's stormwater runoff while the remaining 20% drains directly to the Grand

River. The Kitchener- Waterloo urban area has been heavily industrialized. Major economic activities include furniture, automotive, tire and shoe manufacturing, meat packing and other industries.

DATA COLLECTION PROGRAMS

1 - MOE - COA Program - A data collection program on urban runoff in the Grand River Basin was initiated by the Ontario Ministry of the Environment (MOE) and supported by the Canada-Ontario Agreement on Great Lakes Water Quality (COA) for a duration of two years (1975-1976). Under this program, two urban catchments (the North and West catchments) were established in the City of Guelph and instrumented for quantity (precipitation and runoff) and quality monitoring.

The objectives of the program were to investigate the variation of quantity and quality of stormwater runoff; to document the difficulties encountered in the selection and instrumentation of representative urban catchments and, to investigate the applicability of the Storage, Treatment, Overflow and Runoff Model (STORM) for prediction of urban runoff quantity and quality.

The North Catchment is located at the northern limits of the City of Guelph (Figure 4). The catchment is a relatively new suburban division consisting primarily of single family residential land use (82.9%) with minor areas of multiple family residential (3.5%), commercial and institutional (6.2%) and open space (3.3%). The area of the catchment is 58.8 ha with a total imperviousness of 39.0%.

The West Catchment (Figure 4) is adjacent to the downtown core of the City of Guelph. The catchment (234.8 ha) represents a range of various land uses, old and recently developed, consisting of single and multiple family residential (44.9%), commercial (3.1%), industrial 29.5% and open space 22.5%. The total imperviousness of the catchment was estimated at 32.3% (Novak, in press).

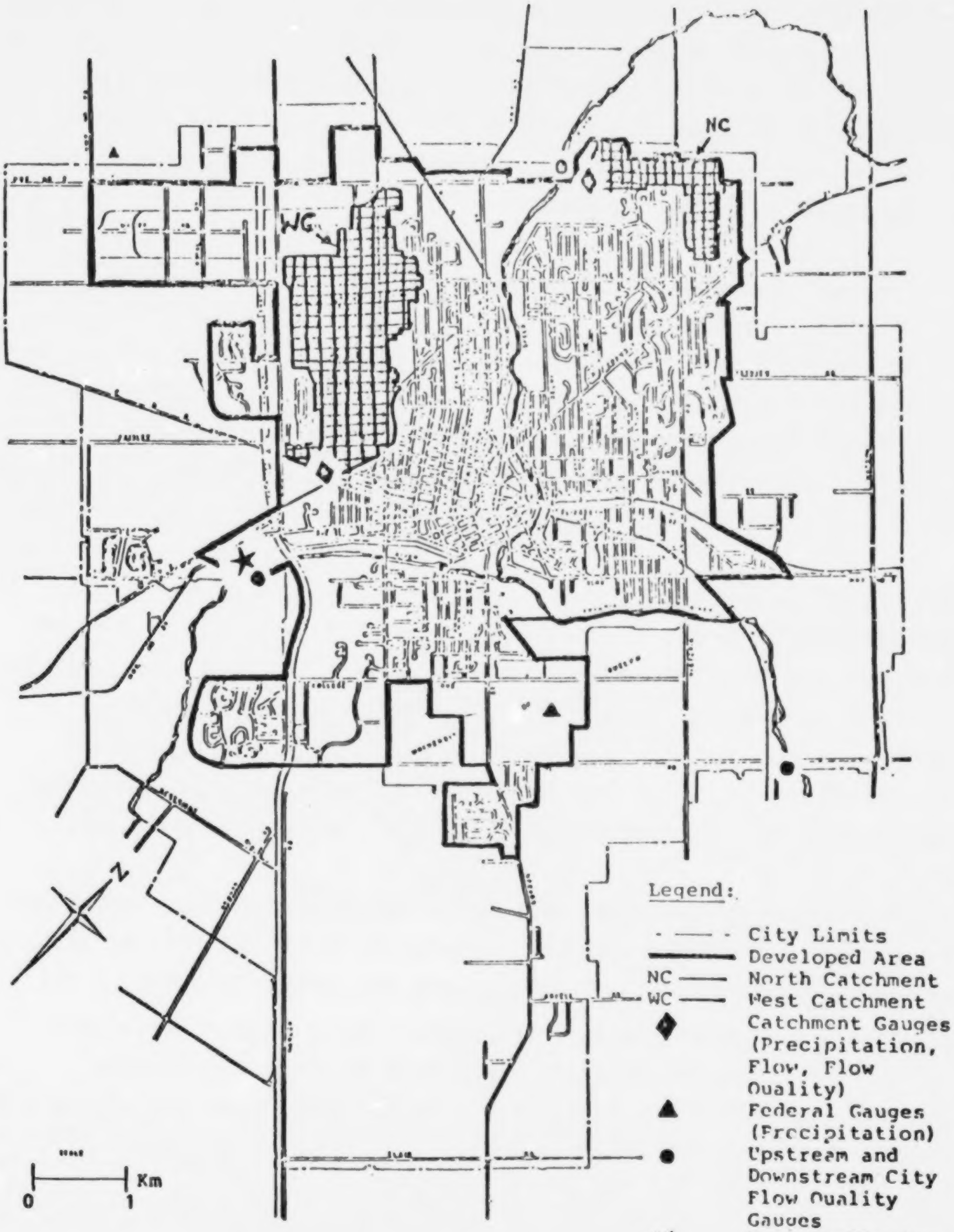
NC
WC
N
Km
0
1
Legend:
City Limits
Developed Area
NC North Catchment
WC West Catchment
Catchment Gauges (Precipitation, Flow, Flow Quality)
Federal Gauges (Precipitation)
Upstream and Downstream City Flow Quality Gauges
Water Pollution Control Plant

Figure 4.

Location of the Northern and Western Urban Catchments and Quantity and Quality Monitoring Stations in the City of Guelph (after Novak, 1979).

Precipitation in both catchments was measured using two Leopold and Stevens tipping bucket rain gauges of capacity of 0.254 mm. Flow measurements were made at the outlet of both catchments and samples were collected manually (during the first phase of the program) and using a manually activated automatic sampler (during the second phase of the program). Samples were analysed for the following parameters:

five-day biochemical oxygen demand,
chemical oxygen demand,
solids (total and suspended),
phosphorus (total and soluble),
nitrogen forms (free ammonia, Kjeldahl, nitrite and nitrate),
iron, lead, phenols, chloride, conductivity and coliforms
(total and fecal) and streptococcus sp.

During this program a total of 14 events were monitored in the North Catchment and 13 events in the West Catchment. Unfortunately, only four events in the North Catchment and two in the West Catchment were completely monitored in terms of precipitation, flow (hydrograph) and quality (pollutograph). The remaining events were partially monitored. This program clearly indicated that automatic sampling stations are essential for the successful conduct of any urban runoff quality investigation.

2 - PLUARG Program - A second data collection program on urban runoff was established in the Grand River Basin under the Pollution from Land Use Activities Reference Group (PLUARG) during the period (1975-1977). The objectives of this program were to determine the sources of pollutants within urban areas, estimate their magnitude in terms of unit-area loadings, determine their relative significance and investigate the nature of their transport from urban areas.

Three urban areas were selected in the Grand River Basin for the study, namely: the megalopolis of Kitchener-Waterloo-Cambridge, the City of Guelph and the Town of New Hamburg. In addition, two small urban watersheds were monitored, namely: Schneider Creek and Montgomery Creek. Monitoring stations were established upstream and downstream of the urban areas for the purpose of collecting flow and quality information for pollutant loading estimates (Figure 5). The difference between the pollutant load measured at the outlet (downstream) station, and the inflow (upstream) stations was considered to be the net pollutant load from the study area. The load from the urban sources was then estimated by subtracting measured point sources and other non-urban diffuse sources from the net pollutant load (O'Neill, 1979).

3 - The Grand River Basin Water Management Study Program - During the Grand River Basin Water Management Study monitoring of the quantity and quality of urban runoff continued in the Schneider Creek and Montgomery Creek watersheds. Flow data were obtained at the outlets of both watersheds and stream water samples were collected using two automatic samplers. Details of flow measurements, sample collection techniques, handling and analytical procedures are discussed in detail in the methodology report (Onn, 1980).

Schneider Creek drains the western portion of the City of Kitchener (Figure 5). The drainage area is 3577 ha and consists of 60% urban, 35% agricultural and 5% wooded land. The major land uses in the urban area are: residential (primarily single family dwellings) 42%, commercial 5%, industrial 4%, recreational 8%, and transportation 1%. The watershed has an estimated population of 74,000 and is serviced by separate storm and sanitary sewer systems. The storm sewer system discharges at several locations along Schneider Creek.

Montgomery Creek drains the eastern portion of the City of Kitchener. The watershed has an area of 958 ha which consists of 96% urban and 4% wooded land. The major land uses in the urban area are: residential 64%, recreational 13%, commercial 12%, transportation 6% and industrial 1%. The watershed has a population of 58,000 and is serviced by separate storm and sanitary sewer systems. The storm sewer system empties into Montgomery Creek at several locations.

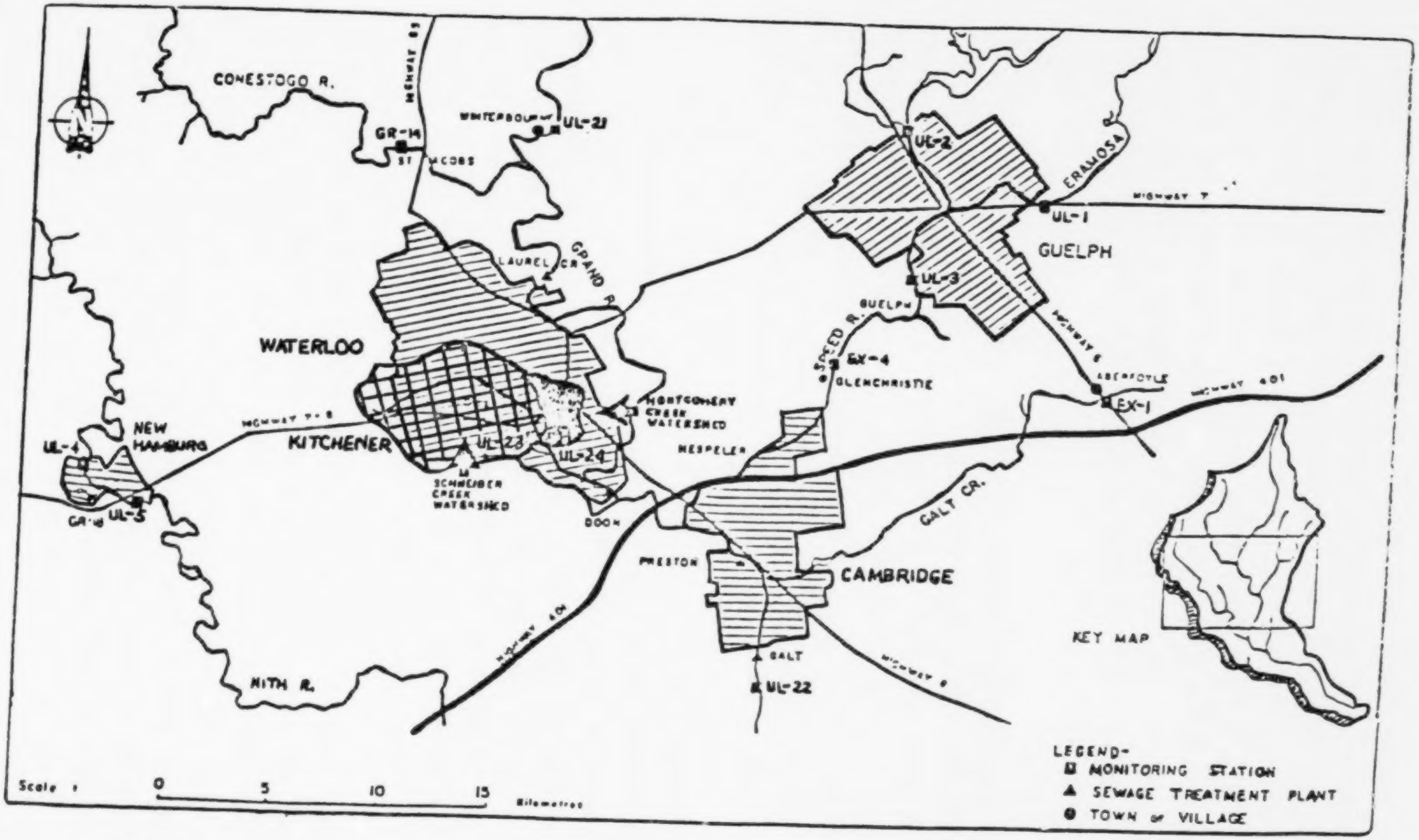
CONESTOGO R.
HIGHWAY 85
GR-14
ST JACOBS
WINTERBOURNE
UL-21
GRAND R.
LAUREL CR.
WATERLOO
KITCHENER
HIGHWAY 7 + 8
NEW HAMBURG
UL-4
UL-5
GR-18
UL-23
UL-24
SCHNEIDER CREEK WATERSHED
MONTGOMERY CREEK WATERSHED
DOON
HESPELER
PRESTON
CAMBRIDGE
GALT
UL-22
GALT CR.
HIGHWAY 8
HIGHWAY 401
NITH R.
SPEED R.
EX-4
GLENCHRISTIE
GUELPH
UL-2
UL-3
UL-1
ERAMOSA R.
HIGHWAY 7
HIGHWAY 6
ABERFOYLE
EX-1
GUELPH
KEY MAP
LEGEND-
▣ MONITORING STATION
▲ SEWAGE TREATMENT PLANT
● TOWN or VILLAGE
Scale :
0
5
10
15
Kilometres
N

Figure 5. Urban Areas in the Grand River Basin Monitored Under the PLUARG Program.

During the monitoring period a total of 22 hydrographs and 10 pollutographs were obtained for Schneider Creek and 14 hydrographs and 4 pollutographs for Montgomery Creek. The pollutographs, based on a sampling frequency ranging from 15 to 30 minutes, were produced for suspended solids, five-day biochemical oxygen demand, total phosphorus, filtered reactive phoshorus, total Kjeldahl nitrogen, filtered ammonia, chloride and lead.

In addition, at the time of initiation of the Grand River Basin Water Management Study in 1975, a network of seven continuous monitoring stations was established to monitor important water quality parameters in the central megalopolis area of the basin (Figure 6). Dissolved oxygen and temperature were monitored continuously at five stations using EIL (Electronic Instruments Ltd.) instrument systems. At two other locations, NERA (New England Research Associates Inc.) instrument systems recorded dissolved oxygen, temperature, conductivity, pH and redox potential at half-hourly intervals (Draper et al, 1980). Data collected at these stations were used to calibrate and verify the Grand River Simulation Model (GRSM) and to assess the impacts of various pollution sources including urban runoff on the water quality of the Grand River.

Characteristics of Urban Runoff

Since the PLUARG monitoring program was established to collect flow quantity and quality data for the estimation of annual pollutant loads, intensive sampling was not conducted at the monitoring stations during each storm event. As such, detailed runoff quality data are available only for a few storm events which occurred in the sampling period. For this reason, the characterization of urban runoff from the four test catchments relied primarily upon data generated from the other two monitoring programs. A statistical summary of the runoff quality data obtained is presented in Table 2.

Table 2 indicates that the quality data collected in each of the four test catchments span several orders of magnitude. These wide variations are in agreement with the characteristics of urban runoff reported for other

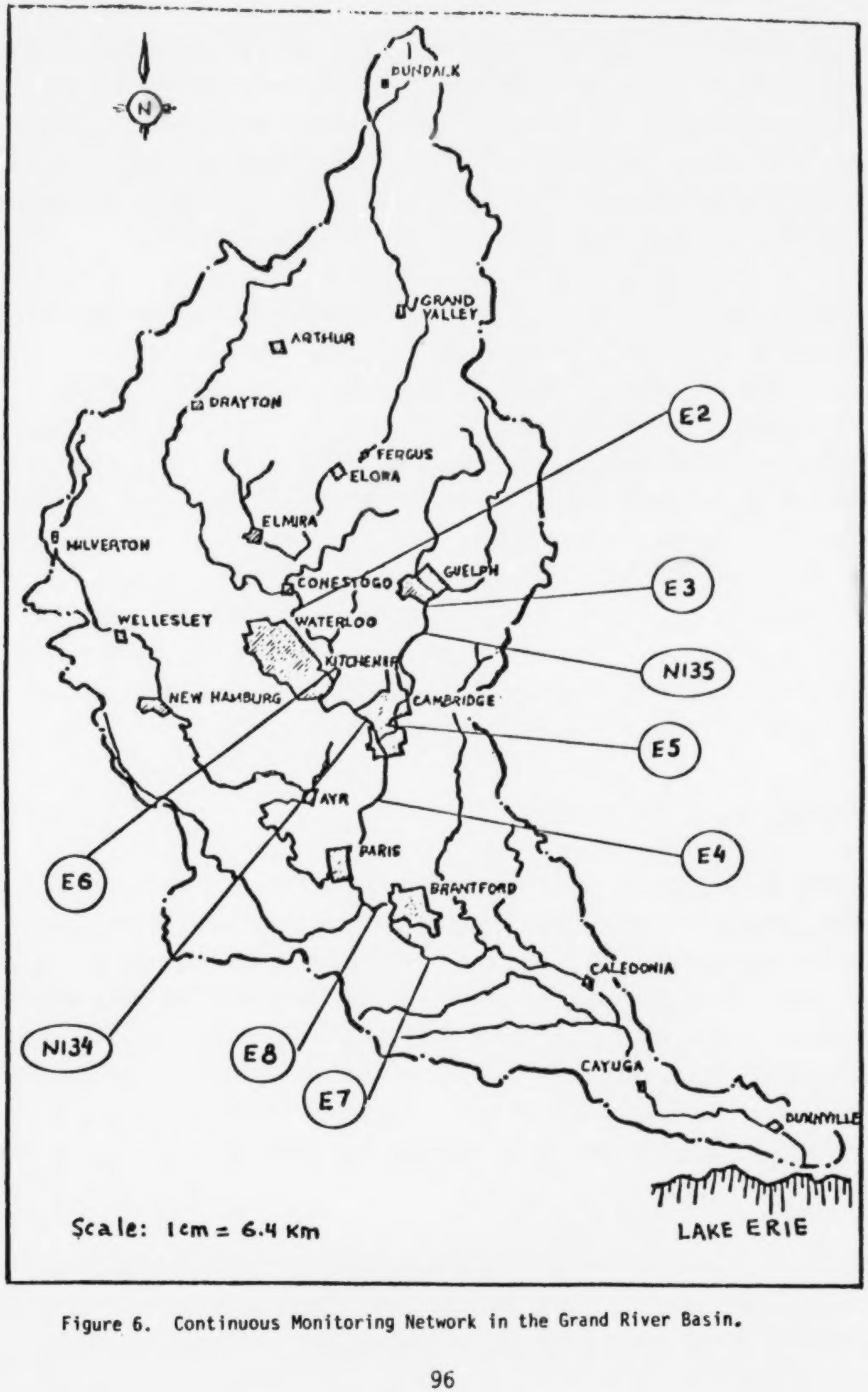
N
DUNDALK
GRAND VALLEY
ARTHUR
DRAYTON
FERGUS
ELORA
ELMIRA
MILVERTON
CONESTOGO
GUELPH
WELLESLEY
WATERLOO
KITCHENER
NEW HAMBURG
CAMBRIDGE
AYR
PARIS
BRANTFORD
CALEDONIA
CAYUGA
DUNNVILLE
E2
E3
N135
E5
E4
E6
N134
E8
E7
Scale: 1cm = 6.4 Km
LAKE ERIE

Figure 6. Continuous Monitoring Network in the Grand River Basin.

TABLE 2

STATISTICAL SUMMARY OF URBAN RUNOFF QUALITY DATA

	Suspended Solids	5-day Biochemical Oxygen Demand	Total Phosphorus	Total Nitrogen	Lead	Zinc	Chloride
1. SCHNEIDER CREEK WATERSHED							
Mean mg/L	267	3.9	0.66	3.04	0.08	0.78	66
Standard Deviation, mg/L	643	2.5	1.10	2.19	0.07	0.71	36
Range, mg/L	5-4791	0.6-13.0	0.07-9.60	1.21-20.15	0.01-0.35	0.02-3.20	10-175
Number of Samples	128	128	127	124	117	117	124
2. MONTGOMERY CREEK WATERSHED							
Mean, mg/L	81	3.1	0.19	2.50	0.14	0.18	61
Standard Deviation, mg/L	65	2.2	0.10	1.21	0.09	0.15	45
Range, mg/L	19-445	0.2-9.5	0.06-0.64	0.96-7.10	0.01-0.40	0.03-1.00	8-188
Number of Samples	87	87	86	86	72	73	86
3. NORTH CATCHMENT, GUELPH							
Mean, mg/L	77	10.2	0.20	2.30	-	-	-
Range, mg/L	10-1090	0.2-60	0.04-1.60	0.40-5.30	-	-	1-68
4. WEST CATCHMENT, GUELPH							
Mean, mg/L	195	13.9	0.35	3.70	-	-	-
Range, mg/L	5-756	0.2-95	0.03-2.40	0.2-3.4	0.01-0.65	-	0-383

cities in Ontario (Weatherbe and Novak, 1977) and can be attributed to the large number of factors known to affect the quality of urban runoff.

It can be seen in Table 2 that the concentrations of suspended solids and total phosphorus were highest in the runoff from the Schneider Creek watershed. The high suspended solids concentration values can be attributed to the construction activities in progress in the watershed during the monitoring period while the elevated concentration of phosphorus can be explained by the attachment of phosphorus on sediment particles. For undetermined reasons, the concentration of five-day biochemical oxygen demand was higher for the two test catchments in Guelph than in the Schneider Creek and Montgomery Creek watershed.

A comparison between the dry-weather and wet-weather data indicates that the concentrations of the monitored water quality parameters increased when streamflow increased. However, only weak correlations were found during storm events between flow rates and the concentrations of the monitored water quality parameters. Typical runoff hydrographs and pollutographs obtained from the Schneider Creek and Montgomery Creek watersheds are shown in Figures 7 and 8. Specifically, the first flush phenomenon was not observed in either watershed though it was noted in the two smaller catchments in Guelph. Apparently the quick hydrologic response of a small catchment favours the occurrence of the first flush phenomenon.

A correlation analysis was performed using the wet-weather water quality data collected from the Schneider Creek and Montgomery Creek watersheds. Relatively strong correlation (coefficient of correlation $r = 0.80$) was found between suspended solids and total phosphorus but not among the other water quality parameters.

The effects of total event precipitation, total runoff volume and length of the antecedent dry period on event pollutant loads were analyzed using data collected in the Schneider Creek watershed. Total event precipitation and total runoff volume were found to be the most dominant factors which determine the event pollutant loads.

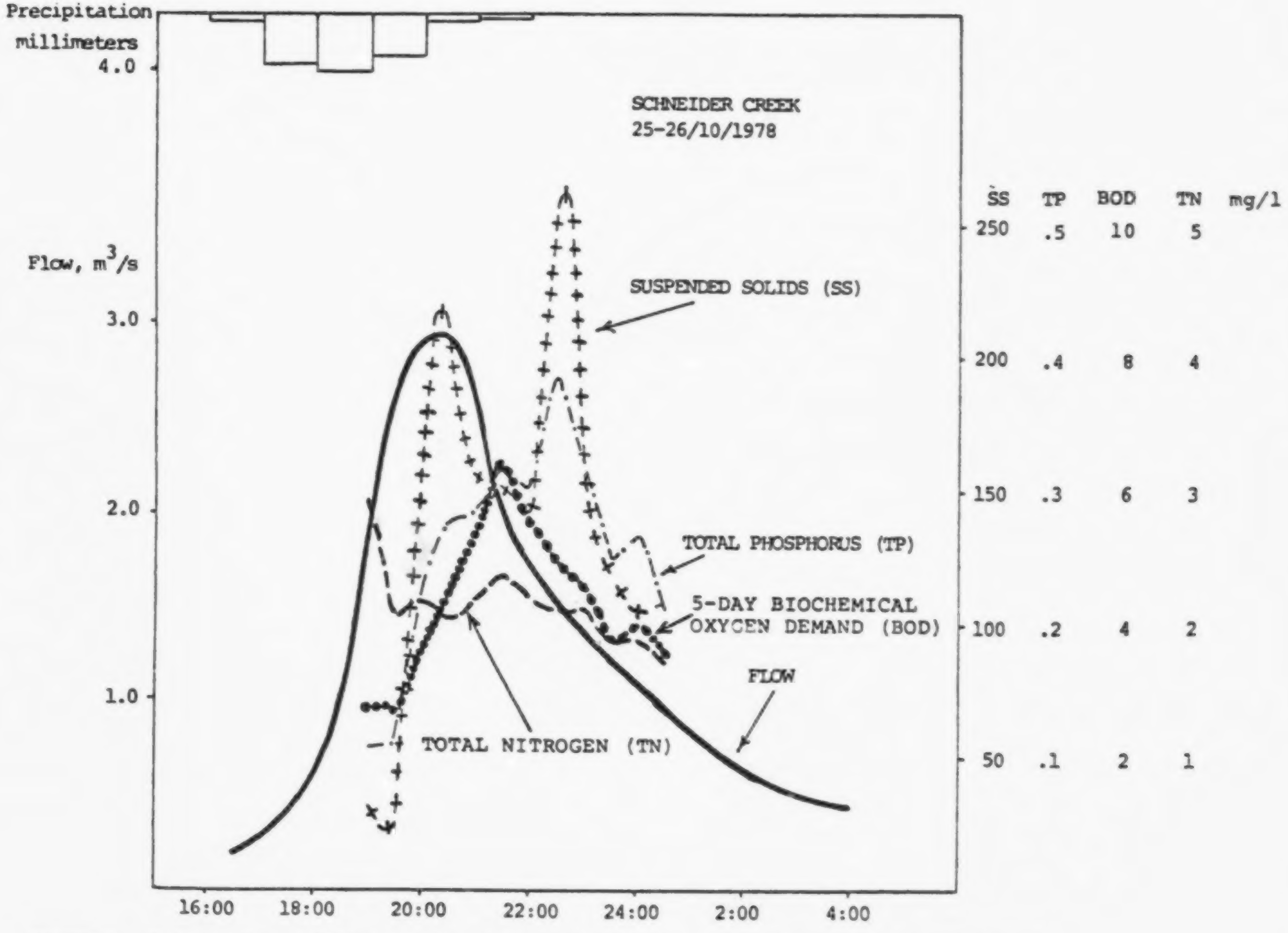

FIGURE 7. Hydrograph and Pollutographs of Suspended Solids, 5-Day Biochemical Oxygen Demand, Total Phosphorus and Total Nitrogen as Measured at the Outlet of Schneider Creek on 25-26/10/1978.

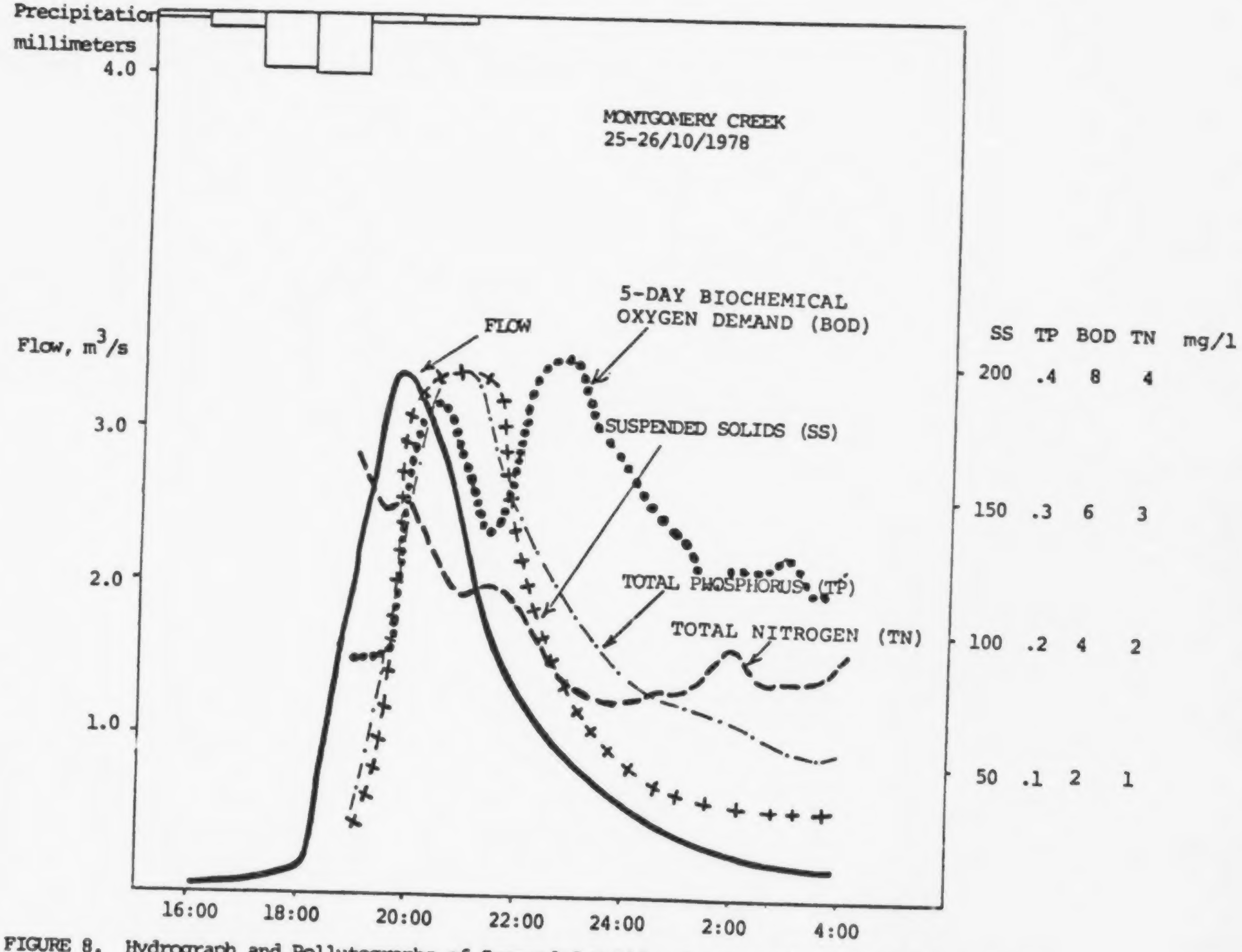

FIGURE 8. Hydrograph and Pollutographs of Suspended Solids, 5-Day Biochemical Oxygen Demand, Total Phosphorus and Total Nitrogen as Measured at the Outlet of Montgomery Creek on 25-26/10/1978.

Simulation of Urban Runoff Quantity and Quality

The two objectives of urban runoff simulation are:

1. to provide input data to the GRSM model in terms of urban runoff volumes and pollutant loads on an event basis from the cities of Brantford, Cambridge, Guelph, Kitchener and Waterloo.

2. to extrapolate urban runoff pollutant loads for the populations projected for the five cities to the years 2001 and 2031.

An evaluation of several urban runoff models with respect to the above objectives lead to the selection of the STORM model (U.S. Army Corps of Engineers, (1977). STORM can simulate both the quantity and quality of urban runoff. It is designed for use with many years of continuous hourly precipitation records but can be used for individual storm events. The model employs an accounting scheme that, for each storm event, allocates runoff volumes to storage and treatment and notes those volumes exceeding storage or treatment capacities. Water quality is handled as a function of hourly runoff rates, with generated quantities of pollutants allocated to storage, treatment or release into receiving waters. Statistics are generated for each event and collectively for all events processed, including average annual values for runoff and pollutant loadings. The results of model calibration and verification, with respect to both runoff quantity and quality, are presented in the following sections.

Runoff Quantity

Fourteen storm events recorded in the Montgomery Creek watershed were used to calibrate the STORM model with respect to total event runoff volume.

The results are illustrated in Figure 9.

The simulation results compare well with the measured event runoff volumes. About 79% of the deviations - the difference between simulated and measured runoff volumes - are less than 25% of the measured runoff volume. Most of the large deviations are associated with minor storm events for which measurement errors are large relative to the event runoff volume.

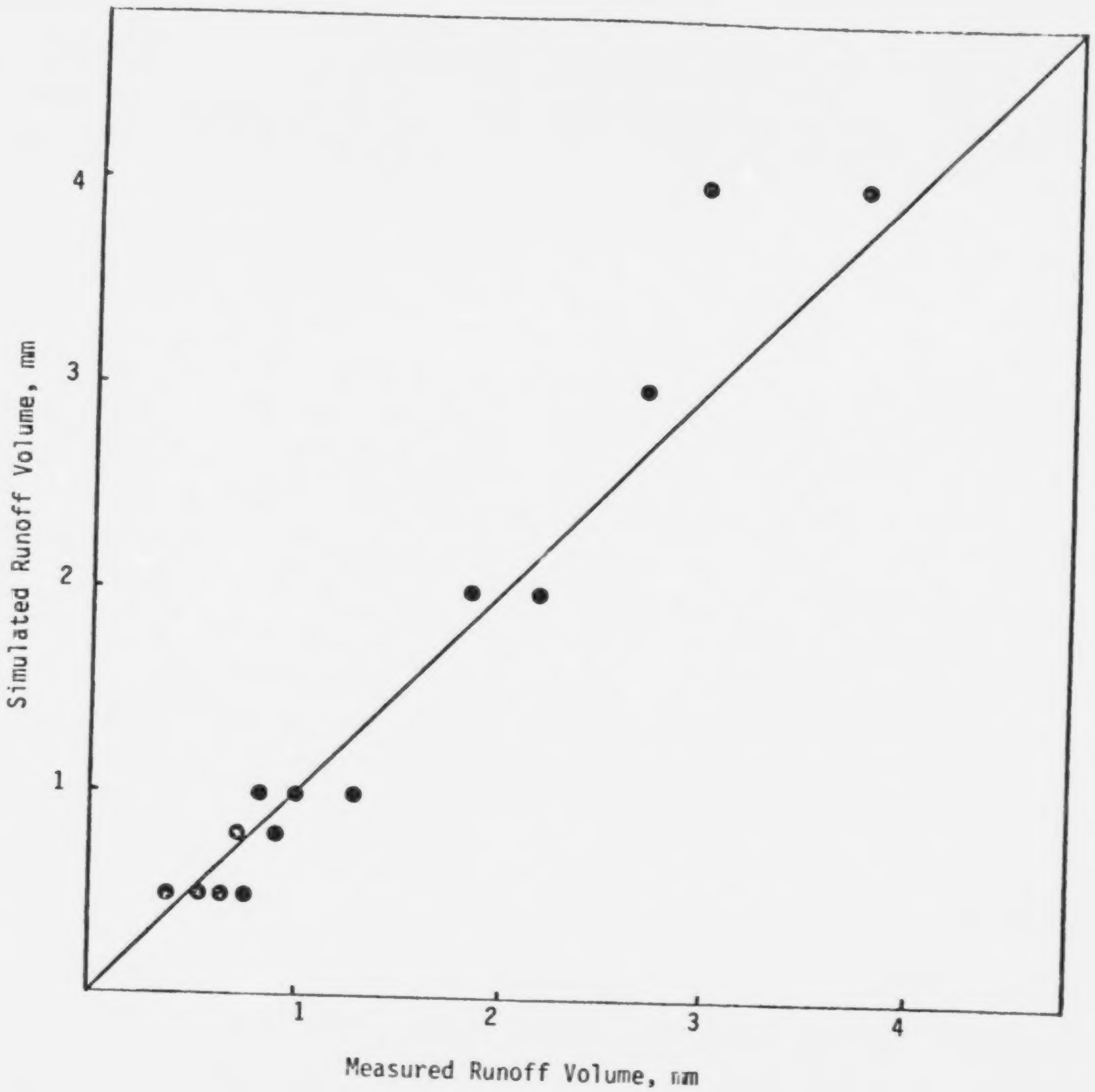

Figure 9. STORM Model Calibration with respect to Event Runoff Volume, Montgomery Creek Watershed.

The coefficients of imperviousness assigned to the various urban land uses were identified to be the most significant factors which determine the event runoff volume. In order to obtain a good agreement between simulated and measured event runoff volumes, it was found necessary to use relatively low imperviousness values. The coefficient of imperviousness was reduced to 15% for residential land use. This low imperviousness value appears to be reasonable since most of the roof leaders in the Montgomery Creek watershed are not connected directly to the storm sewer system.

For model verification, the calibrated model for the Montgomery Creek watershed was used to simulate the quantity of urban runoff from the Schneider Creek Catchment. The calibrated model was adjusted to reflect the characteristics and urban land use distribution in the Schneider Creek watershed. The simulation results (Figure 10) are in close agreement with the measured event runoff volumes. About 50% of the deviations are less than 25% of the measured runoff volume.

Runoff Quality

Although runoff quality data were collected for most of the storm events recorded in the Montgomery Creek watershed (14 events), complete pollutographs are available for only four events. These events were used to calibrate the STORM model in terms of event pollutant loads. The results are given in Table 3.

The results are acceptable, particularly when one considers the simplistic approach used by the STORM model to simulate the quality of urban runoff. In most cases, the simulated event pollutant loads are within ± 50% of the measured loads.

During model calibration, it was found that the simulated event pollutant load depends primarily on three sets of parameters: the pollutant accumulation rates, the characteristics of dust and dirt and the washoff exponent. The effect of the washoff exponent on the shape of the simulated pollutograph, however, was found to be small.

The calibrated model for the Montgomery Creek watershed was used to simulate the quality of urban runoff from the Schneider Creek watershed. As mentioned earlier, the model was adjusted to reflect the characteristics and urban land use distribution in the Schneider Creek watershed. The measured and simulated event pollutant loads are compared in Table 4.

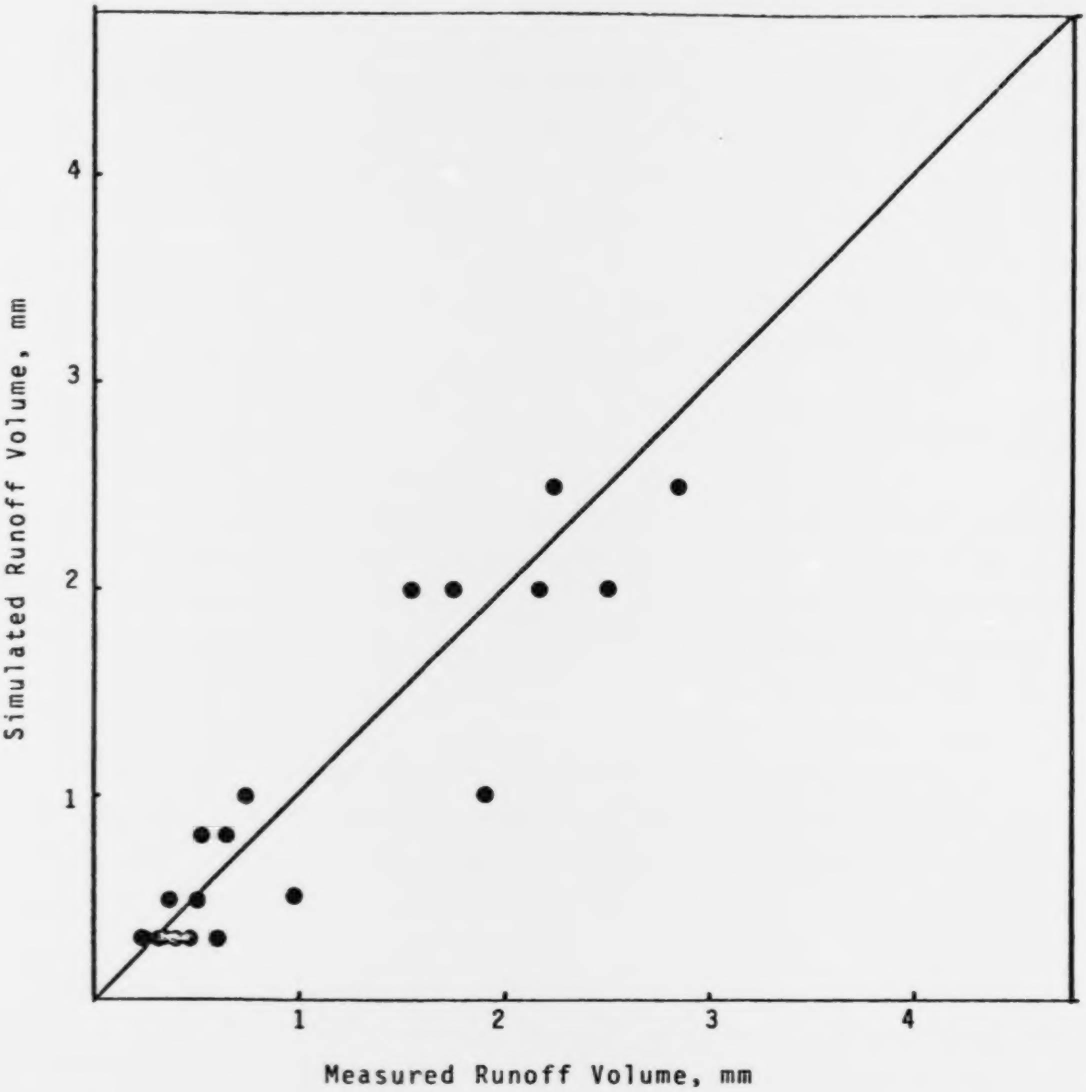

Figure 10. STORM Model Verification with respect to Event Runoff Volume, Schneider Creek Watershed.

TABLE 3

STORM MODEL CALIBRATION WITH RESPECT TO EVENT POLLUTANT LOADS, MONTGOMERY CREEK CATCHMENT

Date Storm Event Occurred	Suspended Solids kg		5-day Biochemical Oxygen Demand kg		Total Phosphorus kg		Total Nitrogen kg	
	Measured	Simulated	Measured	Simulated	Measured	Simulated	Measured	Simulated
3/10/78	1909	2849	109	178	5.3	7.0	120	151
5/10/78	176	296	8	17	0.6	0.8	32	15
25/10/78	1619	1362	78	93	3.3	3.3	66	73
23/11/78	2768	2889	26	167	5.4	7.1	158	151

TABLE 4

STORM MODEL VERIFICATION WITH RESPECT TO EVENT POLLUTANT LOADS, SCHNEIDER CREEK CATCHMENT

Date Storm Event Occurred	Suspended Solids kg		5-day Biochemical Oxygen Demand kg		Total Phosphorus kg		Total Nitrogen kg	
	Measured	Simulated	Measured	Simulated	Measured	Simulated	Measured	Simulated
16/8/78	906	2711	72	186	6.3	6.7	112	146
24/8/78	2033	1800	106	120	5.5	4.4	104	97
14/9/78	18383	12781	254	703	43.3	31.6	390	661
17/9/78	372254	104926	1001	5006	857.0	258.8	3592	5307
30/9/78	1684	670	47	39	2.6	1.7	46	35
3/10/78	21924	8786	453	489	48.2	21.6	434	459
5-6/10/78	1843	2513	24	132	4.7	6.2	106	130
25/10/78	5750	5613	171	325	10.5	13.9	182	295
13/11/78	12473	7522	488	478	26.6	18.6	392	399
23/11/78	21268	6950	71	384	30.2	17.1	376	361

On the whole, the simulated event pollutant loads compare well with the measured loads; both are of the same order of magnitude. The results are particularly good for total phosphorus and total nitrogen; the simulated event loads are within ± 50% of the measured loads. The results are not as good for suspended solids and five-day biochemical oxygen demand.

Long-Term Simulation

The STORM model was used to simulate the urban runoff and associated pollution loads for the cities of Brantford, Cambridge, Guelph, Kitchener and Waterloo for a 20-year period (1956-1975). For each city, the model was adjusted to reflect the characteristics and urban land use distribution. Parameter values for the pollutant accumulation rates, the characteristics of dust and dirt and the washoff exponent were the same as used in the calibrated model. A statistical summary of the simulation results is presented in Table 5. Simulated urban runoff volumes and loads for the five cities were used as input to GRSM model for impact assessment.

TABLE 5 STATISTICAL SUMMARY OF SIMULATION RESULTS (1956 - 1975)

	Brantford	Cambridge	Guelph	Kitchener	Waterloo
Area, ha	7943	9080	7611	12954	6477
Runoff as Fraction of Precipitation	0.17	0.19	0.19	0.15	0.15
Pollutant Loads, kg/ha/yr					
Suspended Solids	92	108	87	75	75
5-day Biochemical Oxygen Demand	4.8	5.6	4.5	3.9	3.9
Total Phosphorus	0.23	0.27	0.22	0.18	0.18
Total Nitrogen	9.5	11.1	9.0	7.7	7.7

A comparison between the annual pollution loads of suspended solids, total phosphorus and total nitrogen from the five urban centres and from agricultural and sewage treatment plants within the Grand River Basin indicates that the urban percentage contribution is small (2%-6%).

IMPACT OF URBAN STORMWATER RUNOFF ON THE GRAND RIVER

Untreated stormwater runoff may contribute a significant portion of the total pollution load entering receiving waters on an annual basis, and are often significant on a shock-load basis during wet events.

When pollutants from urban runoff are discharged into receiving waters, they may affect the water quality in several ways. Some of their effects are immediate such as bacteria contamination. Others are long-term effects such as nutrient enrichment which may lead to eutrophication.

Receiving waters such as streams, lakes and estuaries differ in the manner in which they react to similar pollutant loadings. Further, the types, extents and rates of water quality processes that occur in water bodies are controlled by the immediate physical environment as defined by climate and physiography.

The response of a receiving water to an introduced waste load depends also on its initial state. Thus, the particular response of the receiver under different initial states is basically a matter of defining the appropriate boundary conditions at the time the waste load is imposed.

The impacts of urban runoff can be also viewed in terms of major pollutants (oxygen demanding materials, suspended solids and associated contaminants, nutrients, heavy metals and bacteria) and their specific effects on the various uses of the receiving water (Singer, 1979). For example, suspended solids which find their way into a river may be deposited and become sediment. Sediment is a nuisance if deposited in navigation channels and it can reduce the capacity of drainage ways to carry high flows. Bacterial loadings from urban areas may constitute a health hazard and result in restrictions on swimming and other recreational activities.

All the above considerations indicate that the question of impacts of urban runoff on receiving waters is an issue which is dependent on specific local conditions. For the purpose of the Grand River Basin Water Management study it was decided to investigate the urban runoff impacts on the river in terms of dissolved oxygen (DO).

The strategy was:

1 - to develop a continuous dynamic water quality model capable of accepting inputs from point and nonpoint (urban and rural) sources and predicting the water quality parameters (mainly DO) under various flow regimes, sewage treatment levels and meteorological conditions.

2 - to collect continuous data on DO, temperature and other quality parameters in the Grand River Basin in order to calibrate and verify the model.

3 - to use the model in the assessment of the impacts of various inputs including urban runoff and to evaluate water management alternatives in the megalopolis area of the Grand River Basin.

The Grand River Simulation Model (GRSM)

This is a dynamic model which utilizes O'Connor and DiToro's formulation for the calculation of DO in river systems. The model accounts for the deficits of DO caused by carbonaceous (BOD) and nitrogenous (NOD) oxygen demand, benthic oxygen demand as well as the replenishment of oxygen due to reaeration. Oxygen production and uptake (photosynthesis and respiration) as well as the day to day and seasonal growth, death and washout of three types of attached aquatic plants are calculated using an ecological subroutine (ECOL) (Kwong *et al*, 1979).

The water quality parameters include DO, BOD, NOD, nitrate, suspended solids and total phosphorus. The dynamic model simulates the effects of sewage treatment plants effluent and urban runoff under different flow conditions. The urban loadings are calculated by STORM and input to the dynamic model at five nodes representing the five urban areas: Brantford, Cambridge, Guelph, Kitchener and Waterloo, (Figure 11). Output of the model provides DO concentrations at each 2-hour time step.

GRSM Model Calibration and Verification

The GRSM model was calibrated using actual meteorological data, river hydrology, STP effluents and upstream boundary conditions (quality and quantity) for the year 1976, from June to September. The simulated DO concentrations were compared with the continuous monitoring data for reach No.8 at Glen Christie below Guelph, reach No.12 at Preston and reach No.16 at Glen Morris below Galt STP. Model parameters were adjusted until good agreement was obtained between observed and simulated DO values (Figures 12, 13 and 14).

Actual survey data of 1977 were then used to verify the dynamic model. The verification results indicate that the model is capable of reproducing the observed daily minimum DO within ±1 mg/L approximately 80% of the time. DO frequency distributions at specified levels within a month as a percentage of total time were generally predicted within 10 percentage points of the total time. In general, the accuracy of the predicted minimum DO concentrations is much better than the predicted maximum (Kwong, 1980).

Evaluation of Impact of Urban Runoff

The impact of urban runoff on the Grand River was evaluated by running the GRSM model twice for the period June-September, 1976 The first run included the urban input from the five cities whereas in the second run, the impact of this input was nullified by altering the quality of the stormwater input. This was done to maintain the same flow patterns for the

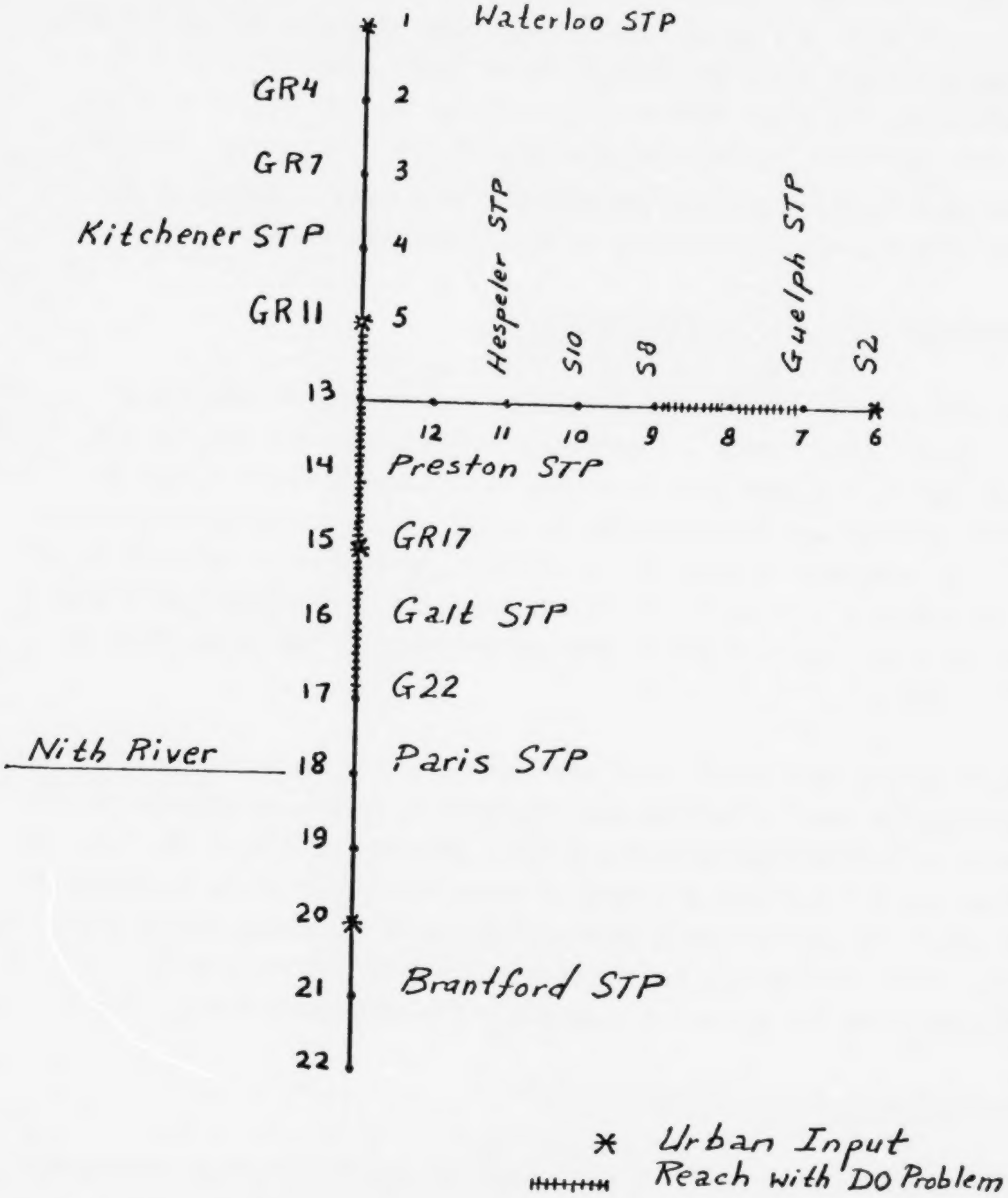
1 Waterloo STP
GR4 2
GR7 3
Kitchener STP 4
GR11 5
Hespeler STP
S10
S8
Guelph STP
S2
13
12
11
10
9
8
7
6
14 Preston STP
15 GR17
16 Galt STP
17 G22
Nith River
18 Paris STP
19
20
21 Brantford STP
22
* Urban Input
Reach with DO Problem

Figure 11. Grand River Water Quality Simulation Model - Geometry of River System.

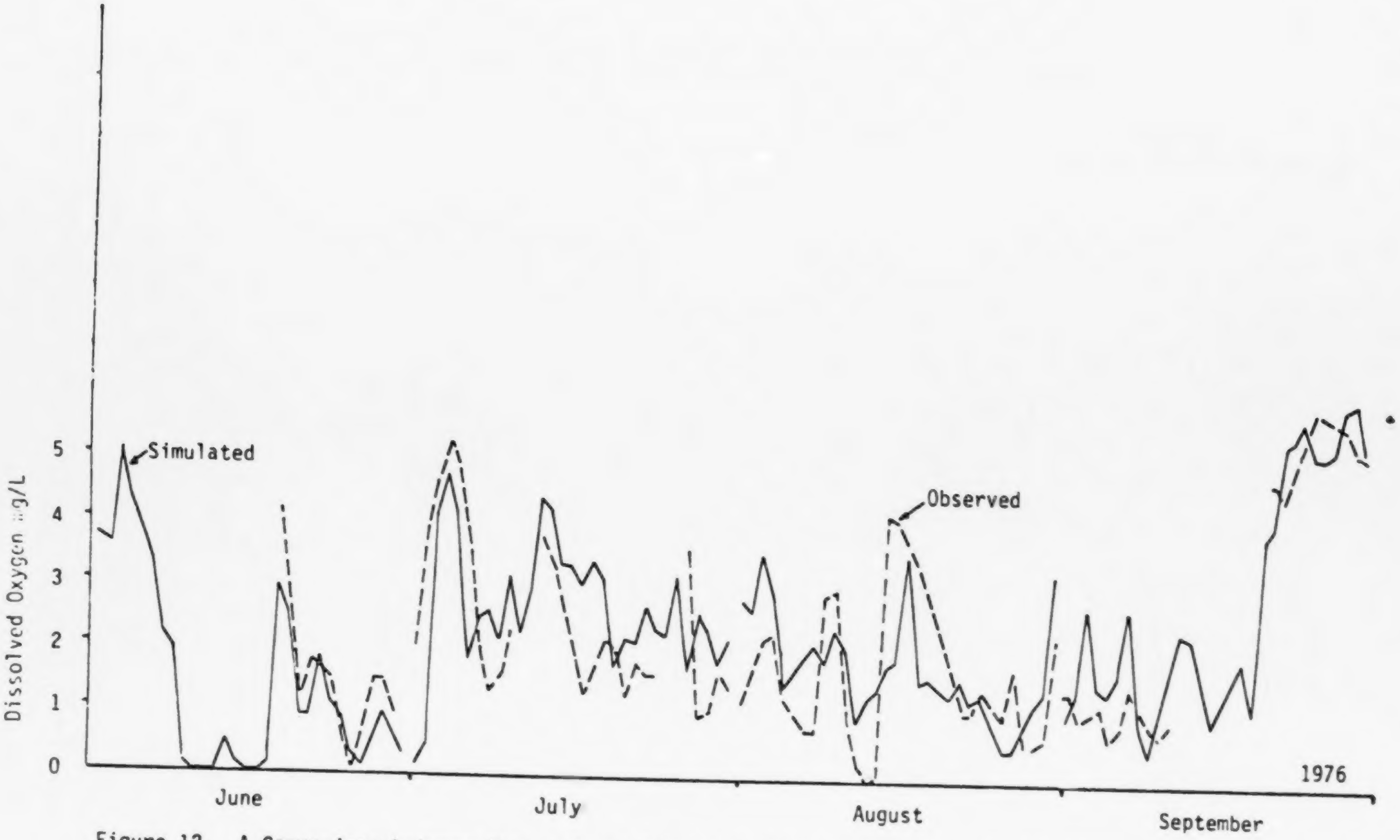

Figure 12. A Comparison between Observed and Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.8 at Glen Christie Below Guelph on the Speed River.

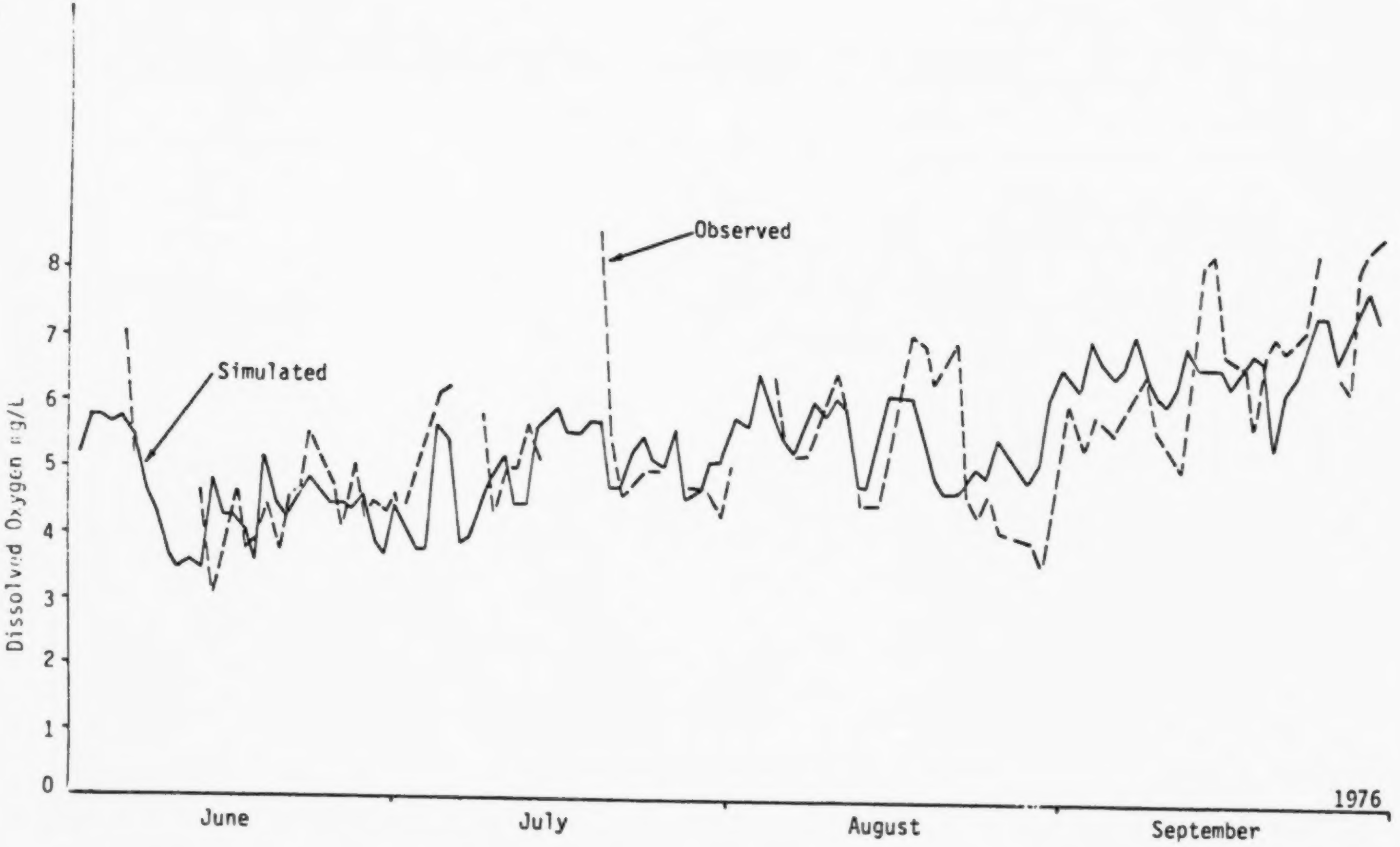

Figure 13. A Comparison Between Observed and Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.12 at Preston on the Speed River.

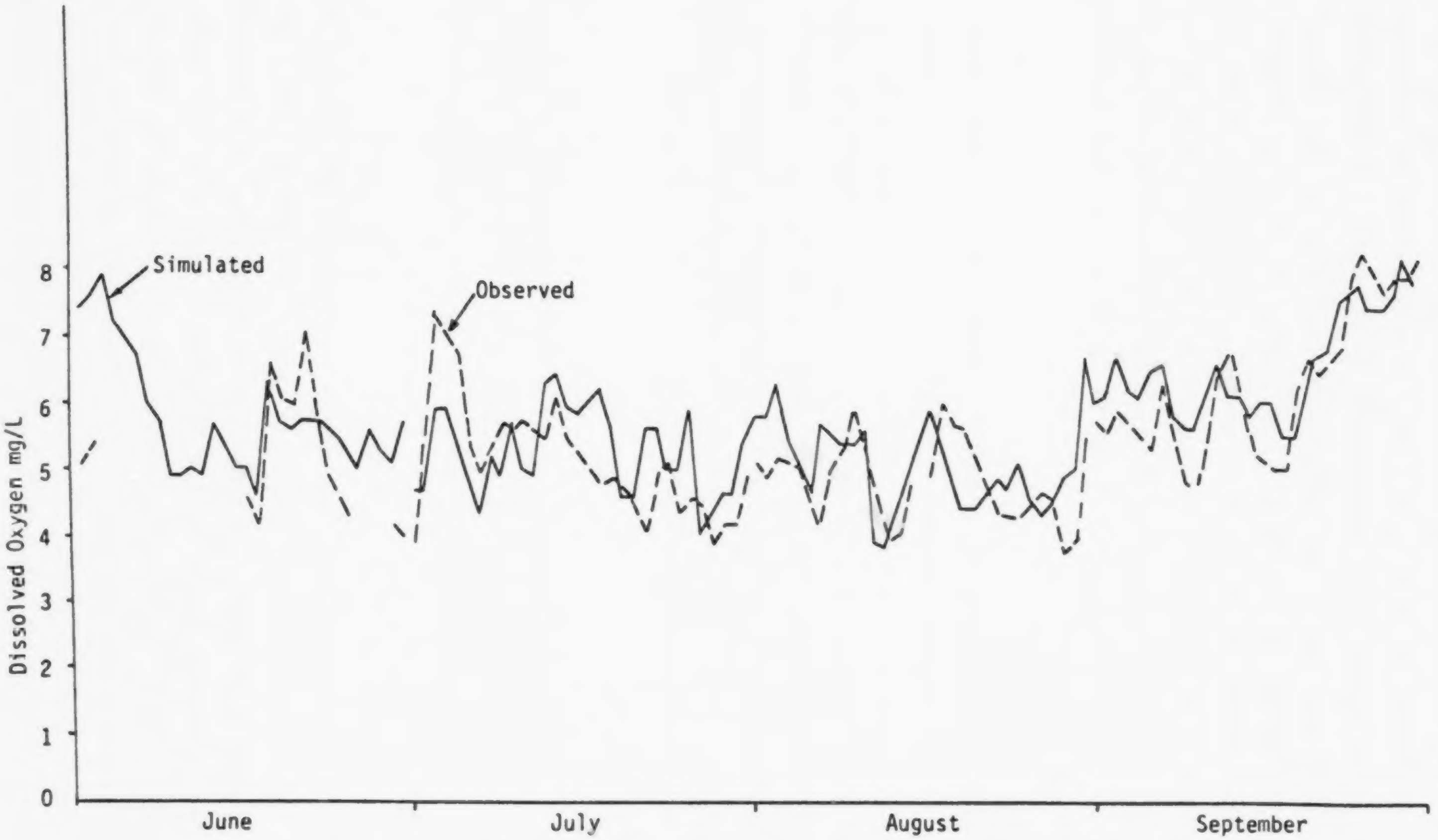

Figure 14. A Comparison Between Observed and Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.16 at Glen Morris Below Galt STP on the Main Grand River.

two runs. Negation of the quality effects of stormwater was achieved by setting the concentrations of BOD, NOD, NO_3, suspended solids and total phosphorus in urban runoff to minimum values of 0.1 mg/L. A comparison of the results of the two simulation runs (Table 6) indicates that the percent of time in DO violations on the Speed River decreased by a few percentage points when urban runoff was excluded. The improvements in the daily minimum DO concentrations (Figures 15 and 16) are minor and average from 0.5 to 1.0 mg/L. The two critical reaches on the Speed River, Reaches No. 7 and 8 were still in violation for 28.1% to 55.9% of the time during the entire simulation period.

The main Grand displayed opposite trends in comparison with the Speed River. The critical reaches, Reach No. 5 (below Kitchener) and Reaches No. 13 - 16, show evidence of an increase in the percent of time in DO violations with the removal of urban input. The worst conditions occurred in Reaches No. 5 and 13 where violations practically doubled. Nevertheless, the changes in the daily minimum DO concentrations averaged less than 1 mg/L (Figure 17). Also, the percent of time in violation for all the reaches on the main Grand ranged from 1.1% to 18.0% for the entire simulation period.

The improvement of the in-stream DO regime of the Speed River with the removal of the urban input is probably due to the reduction of total oxygen demanding load which had a positive effect due to the limited assimilative capacity of this river.

The response of the main Grand to the removal of urban input is hard to explain. It shows that the urban input is having a positive effect on the river under present conditions. A possible explanation of this phenomenon is that the main Grand is light limited and the biomass growth is affected by the available light. By reducing the amount of suspended solids in the urban runoff (second run), the turbidity of the river was reduced and more light was allowed to penetrate to plant depth, thus resulting in more biomass growth and lower DO concentrations. It should be noted, however,

TABLE 6 A COMPARISON BETWEEN THE RESULTS OF TWO SIMULATION RUNS (WITH AND WITHOUT URBAN RUNOFF) IN TERMS OF PERCENT TIME IN DO VIOLATION FOR 21 REACHES ON THE GRAND RIVER (JUNE-SEPTEMBER, 1976)

REACH	JUNE		JULY		AUGUST		SEPTEMBER	
	Run No. 1	Run No. 2	Run No. 1	Run No. 2	Run No. 1	Run No. 2	Run No.1	Run No. 2
1	0.8	0.8	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	3.6	3.6	4.8	5.9	5.4	10.2	0.0	1.4
6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	30.3	28.1	36.8	31.5	43.5	43.5	29.2	28.6
8	50.0	48.6	48.7	41.7	56.5	55.9	44.2	38.3
9	2.5	1.4	7.3	2.7	2.7	5.1	3.9	1.1
10	21.1	19.2	8.6	7.5	0.3	6.5	2.2	3.6
11	1.7	1.1	4.3	1.3	0.0	0.0	1.9	0.0
12	7.5	8.3	3.0	1.1	0.0	0.0	0.0	0.0
13	1.1	1.7	3.2	3.8	4.8	10.2	0.0	0.3
14	1.4	1.4	3.0	4.6	9.4	18.0	0.8	2.2
15	0.0	0.0	2.2	2.4	7.0	11.0	0.0	0.6
16	0.0	0.0	0.0	1.3	0.8	4.8	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

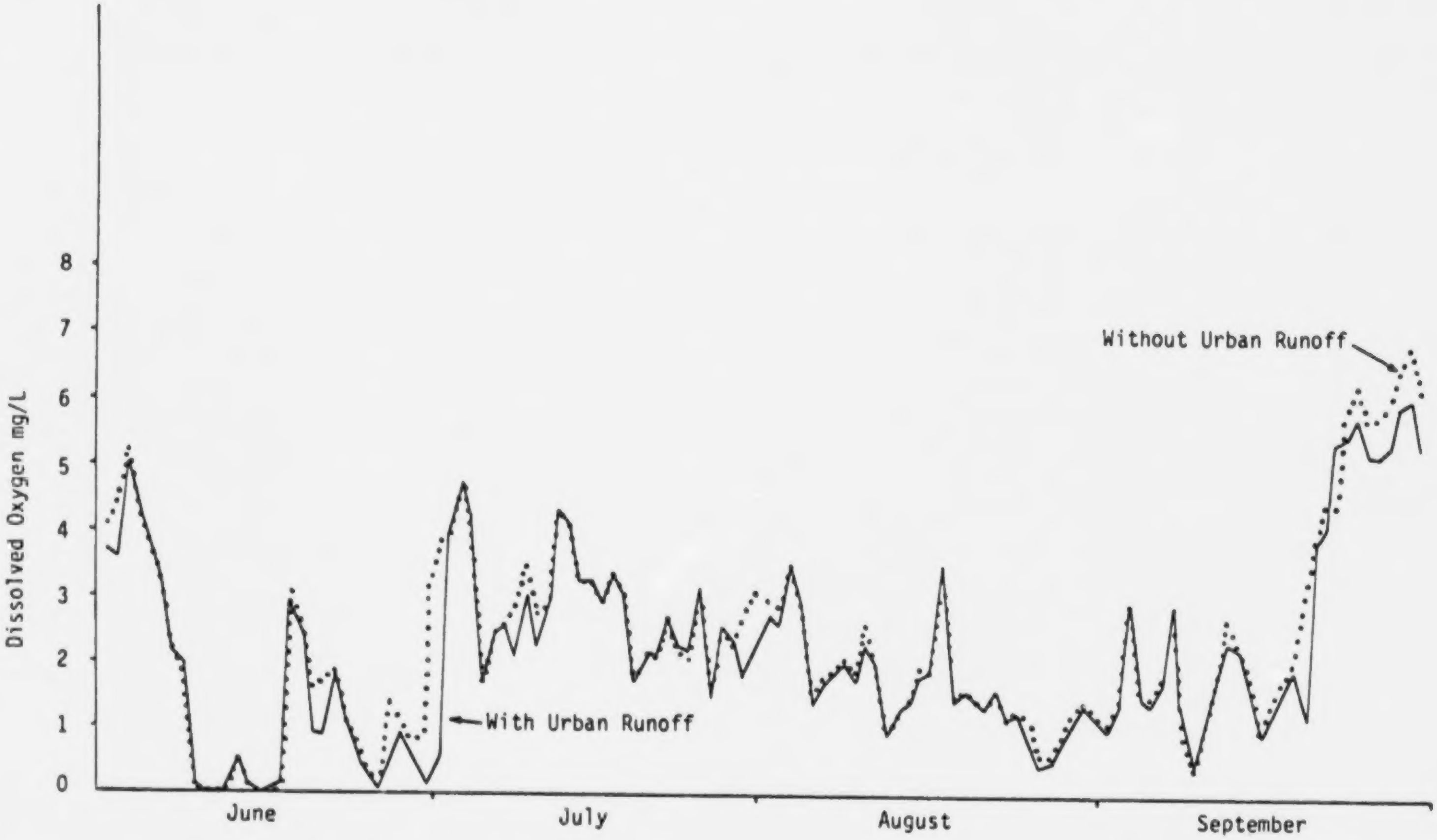

Figure 15. The Impact of Urban Runoff on Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.8 at Glen Christie Below Guelph on the Speed River.

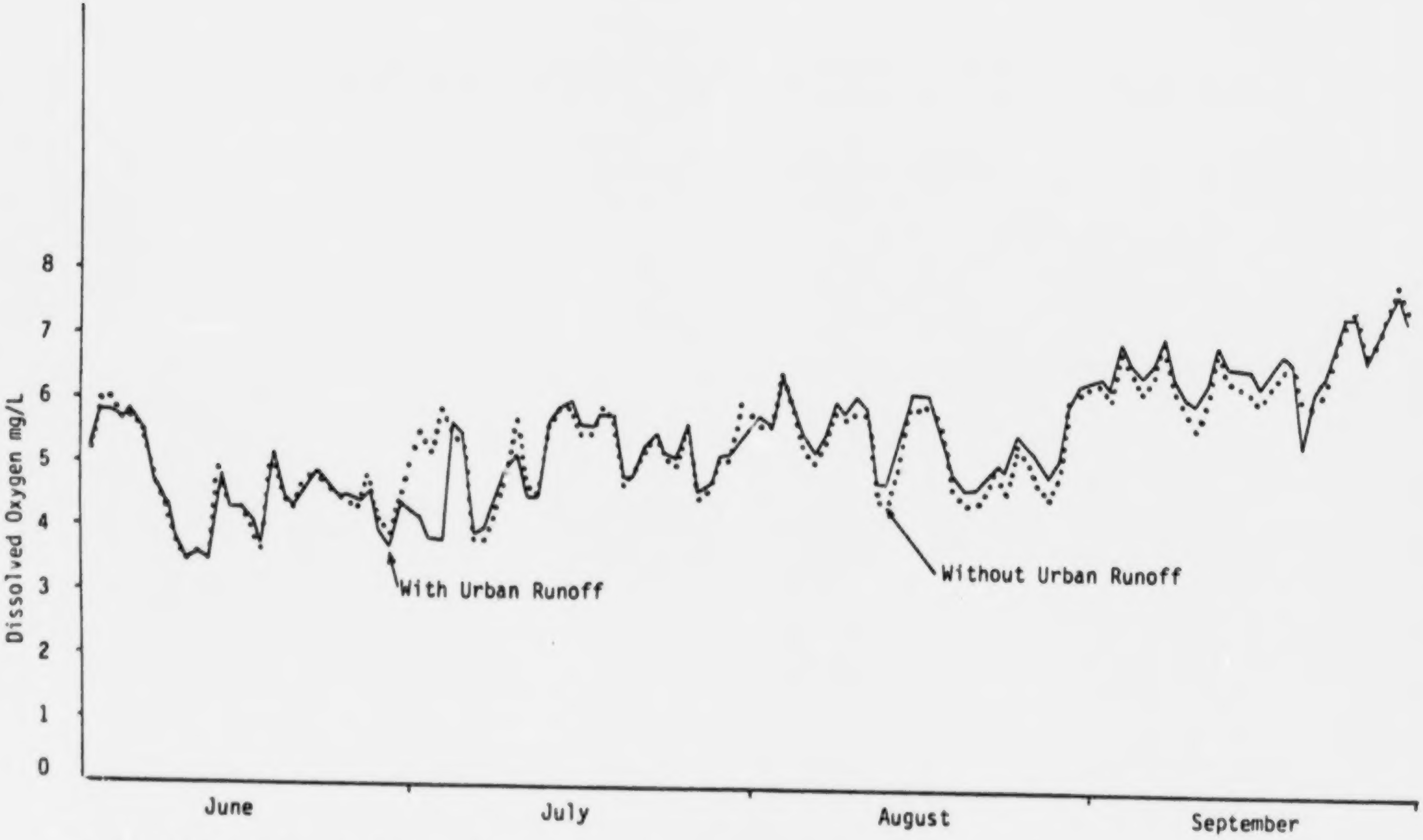

Figure 16. The Impact of Urban Runoff on Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.12 at Preston on the Speed River.

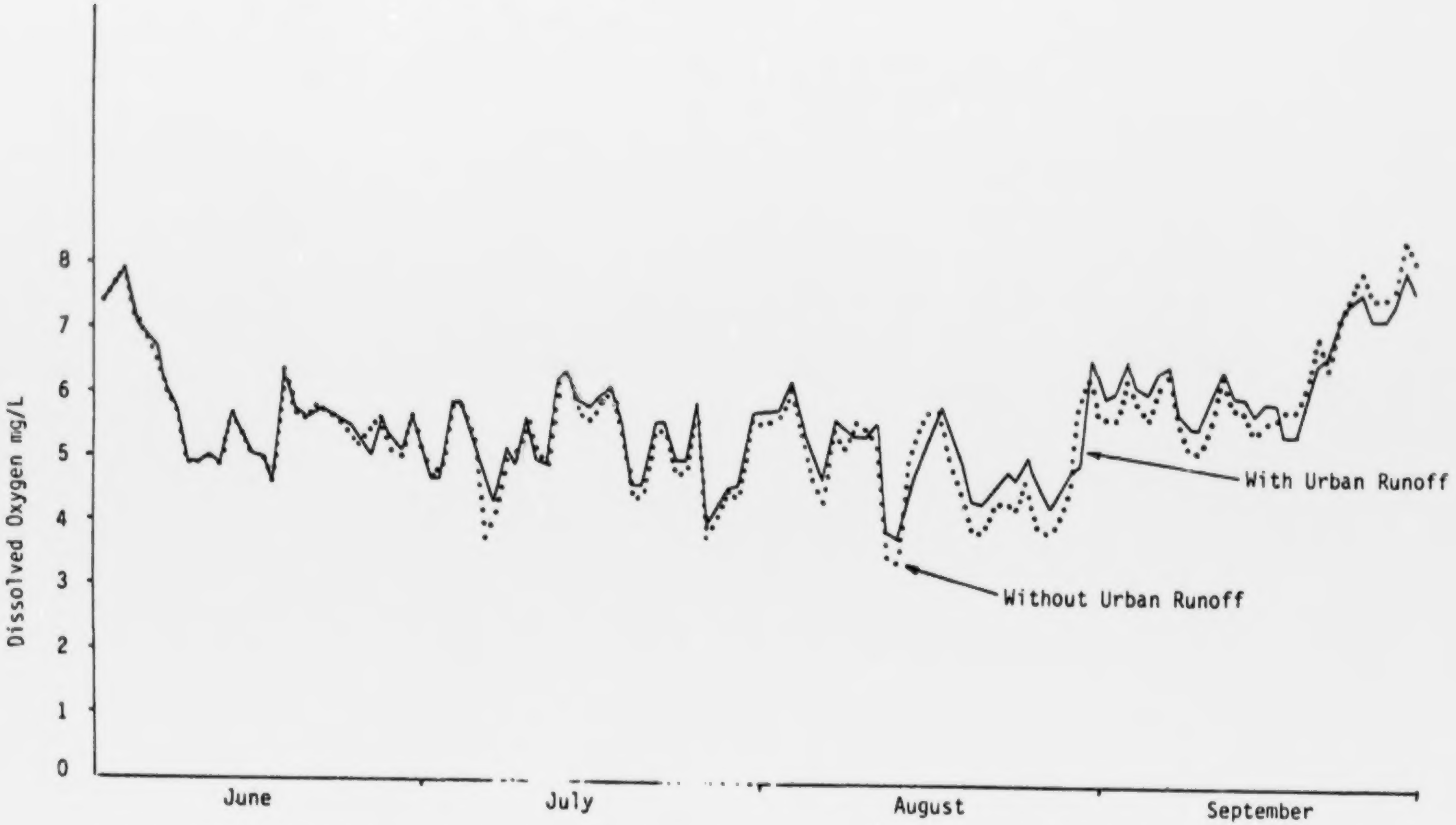

Figure 17. The Impact of Urban Runoff on Simulated Daily Minimum Dissolved Oxygen Concentrations for Reach No.16 at Glen Morris Below Galt STP on the Main Grand River.

that the difference between the two runs in terms of the minimum DO concentrations is minor.

CONCLUSIONS

The characteristics of urban runoff and the magnitude of the associated pollution loads from the cities of Brantford, Cambridge, Guelph, Kitchener and Waterloo are similar to those reported for other cities in Ontario. The impact of urban runoff from the five cities on the dissolved oxygen regime in the Grand River is minor. Parts of the Speed River below Guelph and certain reaches on the main Grand between Kitchener and Brantford suffer from profuse algae and plant growth during the summer and early fall period. This results in extremely low dissolved oxygen concentrations during the night due to the respiration process. High dissolved oxygen levels are observed during the day as a result of the photosynthesis process. Respiration and photosynthesis are the two dominant in-stream processes in this section of the Grand River system. Improvement of the dissolved oxygen regime would require the control of nutrient input (mainly phosphorus) from point and non-point sources. The nutrient input from urban runoff is small relative to agricultural diffuse sources and sewage treatment plants. Therefore priority for pollution control measures should be given to those two sources.

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DEVELOPMENT OF AN URBAN HIGHWAY STORM DRAINAGE MODEL BASED ON SWMM

Raymond J. Dever, Jr.[1] and Larry A. Roesner[2]

BACKGROUND

Water Resources Engineers/Camp Dresser and McKee Inc. is currently completing development of an Urban Highway Storm Drainage Model for the Federal Highway Administration, U.S. Department of Transportation. The general capabilities of the Model include:

- Preliminary highway drainage system design, including locating inlets, sizing pipes, and estimating construction costs;
- Hydraulic analysis of highway drainage systems under rainfall conditions more severe than those used in design; and
- Simulation of the generation and washoff of pollutants in the highway corridor.

The Model consists of four related but independent modules, as follows:

- Precipitation Module
- Hydraulics/Quality Module
- Analysis Module
- Cost Module

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The Precipitation Module can perform a variety of statistical analyses on long-term hourly precipitation data and generate design storm hyetographs. The Hydraulics/Quality Module is the basic design tool in the package. This Module simulates time-varying runoff quantity and quality, locates stormwater inlets and sizes the conduits of the major drainage system. The Analysis Module simulates unsteady gradually-varied flow in the drainage system and can be used to analyze complex hydraulic conditions, such as surcharge and backwater, that may be encountered during an extreme storm event. The Cost Module can be used to estimate construction, operation and maintenance, and total annual costs associated with the drainage system.

The capabilities of each Module will be discussed below in more detail, as will the relationships among them. The Hydraulics/Quality Module employs some of the same formulations that are used in the EPA SWMM package; therefore, this Module will be highlighted, especially with regards to new features that have been added to the basic SWMM routines.

OVERVIEW OF THE HIGHWAY DRAINAGE MODEL

The four Modules of the Urban Highway Storm Drainage Model are a powerful and flexible set of tools for use in drainage system analysis and design. The computer programs which make up the Model have been developed with several key features (1). First, each of the hydraulics programs is fully dynamic. This is in contrast to the static procedure generally used for highway drainage design, where pipes are sized based on a single peak runoff flow generated with the rational formula. Second, the Model has purposely been developed in related but independent Modules to maximize the flexibility of the package. The user may apply as many or as few of the Modules as are appropriate to his particular design or analysis problem. Third, the programs are designed to accommodate local drainage practices and design procedures. The input/output of the programs are in terms familiar to the highway drainage engineer and can include most inlet, channel, and pipe types available. Also, the programs are structured to allow the use of whatever design criteria are required locally.

A summary description of each of the four Modules of the Urban Highway Storm Drainage Model is given below, along with an explanation of the relationships among the Modules.

Precipitation Module

The Precipitation Module consists of a single computer program with three major options - two for generating single-peak synthetic hyetographs and one for performing a variety of statistical analyses on long-term precipitation data (2). The two options for generating synthetic hyetographs both rely on a methodology developed by Chen (3) and require the return frequency, duration, and skew of the desired hyetograph as input. The first option requires the 10-year, 1-hour; the 10-year, 24-hour; and the 100-year, 1-hour rainfalls for the user's study area as input. The second option makes use of an input intensity-duration-frequency curve of the same return frequency as the desired hyetograph. Both of these options can generate the rainfall hyetographs required as input for the dynamic programs of the Hydraulics/Quality Module.

The third major capability of the Precipitation Module is the statistical analysis of long-term hourly precipitation records (such as those available on magnetic tape from the National Oceanic and Atmospheric Administration, U.S. Department of Commerce). Some of the statistical analyses of which the program is capable are:

- Annual series analysis and partial duration series analysis, including the generation of intensity-duration frequency curves;
- Frequency of occurrence analysis for such parameters as peak rainfall intensity per storm event, storm duration, and dry period duration; and

- Analysis of storm skew (i.e., the ratio of the time to peak rainfall intensity of a given storm to the total duration of the storm).

Each of these analyses may be performed on a seasonal basis as well as an annual basis, for as many as 12 user-defined seasons.

The statistical analyses that can be performed with the Precipitation Module clearly give the user of single-event stormwater simulation models better local information than is generally available for developing the required design storm event and associated conditions. For example, the dry period frequency of occurrence analysis should greatly assist the modeler in specifying antecedent conditions for his simulations. The storm skew analysis should enable the engineer to use a rainfall distribution more representative of his local area than some others routinely used, such as the Soil Conservation Service Type-II storm distribution commonly applied to almost the entire continental United States (4).

Hydraulics/Quality Module

The Hydraulics/Quality Module consists of three computer programs - the Inlet Design Program, the Surface Runoff Program, and the Drainage Design Program (5). Together, these programs can perform most of the major computations involved in design of highway drainage systems.

The purpose of the Inlet Design Program is to locate inlets in the surface runoff conveyance system of the highway right-of-way so as to maintain hydraulic conditions during the design storm event within specified criteria. Specifically, the Inlet Design Program simulates time-varying runoff and routes the runoff flows through surface gutters or channels. The program then determines the placement of inlets in gutters required to maintain flow spread within a user-specified maximum or the placement of inlets in channels to maintain flow depth within a user-specified maximum. The program also checks that the percentage of the gutter/channel flow that carries past each inlet to the next gutter/

channel section does not exceed a given maximum, again specified by the user. The routines for computing overland flow and for simulating inlet hydraulics in this program will be discussed in more detail below.

The Surface Runoff Program simulates time-varying runoff quantity and quality, routes these flows and pollutants through surface gutters, channels, and detention basins, and computes inlet hydrographs and pollutographs. The inlet hydrographs and pollutographs are saved as a disc or tape file for subsequent use by the Drainage Design Program. The Surface Runoff Program is similar in most respects to the Runoff Block of EPA's SWMM. Two significant differences between this program and the SWMM Runoff Block are described below.

The Drainage Design Program reads the inlet hydrographs and pollutographs generated by the Surface Runoff Program for the design storm and sizes the major drainage system accordingly. Specifically, the program determines the diameter of circular conduits and the bottom width of trapezoidal channels so that each conduit and channel flows full at peak flow. The diameters of circular conduits so determined are rounded up to the nearest commercially-available pipe size. A complete simulation, including pollutant routing, of the just-designed system is then automatically performed by the program.

Analysis Module

The Analysis Module consists of a single computer program that simulates unsteady, gradually-varied flow in the major drainage system of the highway right-of-way, using inlet hydrographs generated by the Surface Runoff Program as input. The program is basically a modified version of the Extended Transport program of the EPA SWMM package. Its primary purpose is to analyze the performance of the drainage system under an extreme storm event, a step generally included in the highway drainage design process. This program will not be discussed further here, since there are no significant differences between the capabilities of this program and the Extended Transport program.

Cost Estimation Module

The final Module of this package, the Cost Estimation Module, also consists of a single computer program (6). The purpose of this program is to estimate the capital costs, operation and maintenance costs, and total annual costs associated with the construction and maintenance of a highway drainage system. All of the cost computations are based on unit costs for materials, installation, and O&M. As part of the cost analysis, the program also estimates the excavation and backfill associated with construction of the drainage system; elevation of the highway grade line, invert elevations of the system conduits and junctions, and sizes of the conduits and junctions are employed in the excavation and backfill calculations.

DIFFERENCES FROM SWMM

The Hydraulics/Quality Module described above is closely related to the EPA SWMM package in its basic computational routines. However, there are two fundamental differences between SWMM and the Urban Highway Storm Drainage Model that are of interest here - the inclusion of inlet hydraulics and the modification of the overland flow routines to allow cascading of flow over as many as three separate land surface types for each catchment. Both of these modifications are included in the two programs concerned with simulation of surface runoff - the Inlet Design Program and the Surface Runoff Program.

Simulation of Inlet Hydraulics

Both the Inlet Design Program and the Surface Runoff Program are structured to allow the simulation of six basic inlet types:

- Curb Opening Inlet
- Depressed Curb Opening Inlet
- Grate Inlet

- Depressed Grate Inlet
- Combination (Curb Opening and Grate) Inlet
- Depressed Combination Inlet

This is in contrast to the EPA SWMM package in which inlet hydraulics are not simulated at all. The programs are structured to allow simulation of fixed-size inlets of the above six types either by empirical equations built into the program codes or by inlet efficiency curves supplied as input by the user. At present, only empirical equations for depressed curb opening inlets are in the programs, but the codes have been structured to allow easy addition of equations for other inlet types by the user.

The simulation of inlet hydraulics by means of inlet efficiency curves proceeds as follows, in either of the two programs mentioned above. The user supplies as input a group of inlet efficiency curves for the size and type of inlet in question, as shown in Figure 1. (Actually, the user supplies the coordinates of points that define the curves.) The curves give the percentage of gutter or channel flow intercepted by the inlet as a function of the total gutter or channel flow at a given point in time. For inlets in gutters, the flow interception capacity is a function of both the gutter slope and the cross-slope of the highway surface; thus, a family of curves, one curve for each of several typical gutter slopes and cross-slopes, must be supplied. For inlets in channels, the flow interception capacity is a function of the channel slope; a family of curves for typical channel slopes must be input by the user. In the example curves of Figure 1, Q_I is the flow intercepted by the inlet, Q is the gutter/channel flow, and S_x is the cross-slope of the highway.

For a given inlet, the program will select the appropriate inlet efficiency curve to use based on the slope of the gutter/channel section where the inlet is located and the cross-slope of the highway, if appropriate. At each time step, the program then calculates the gutter/

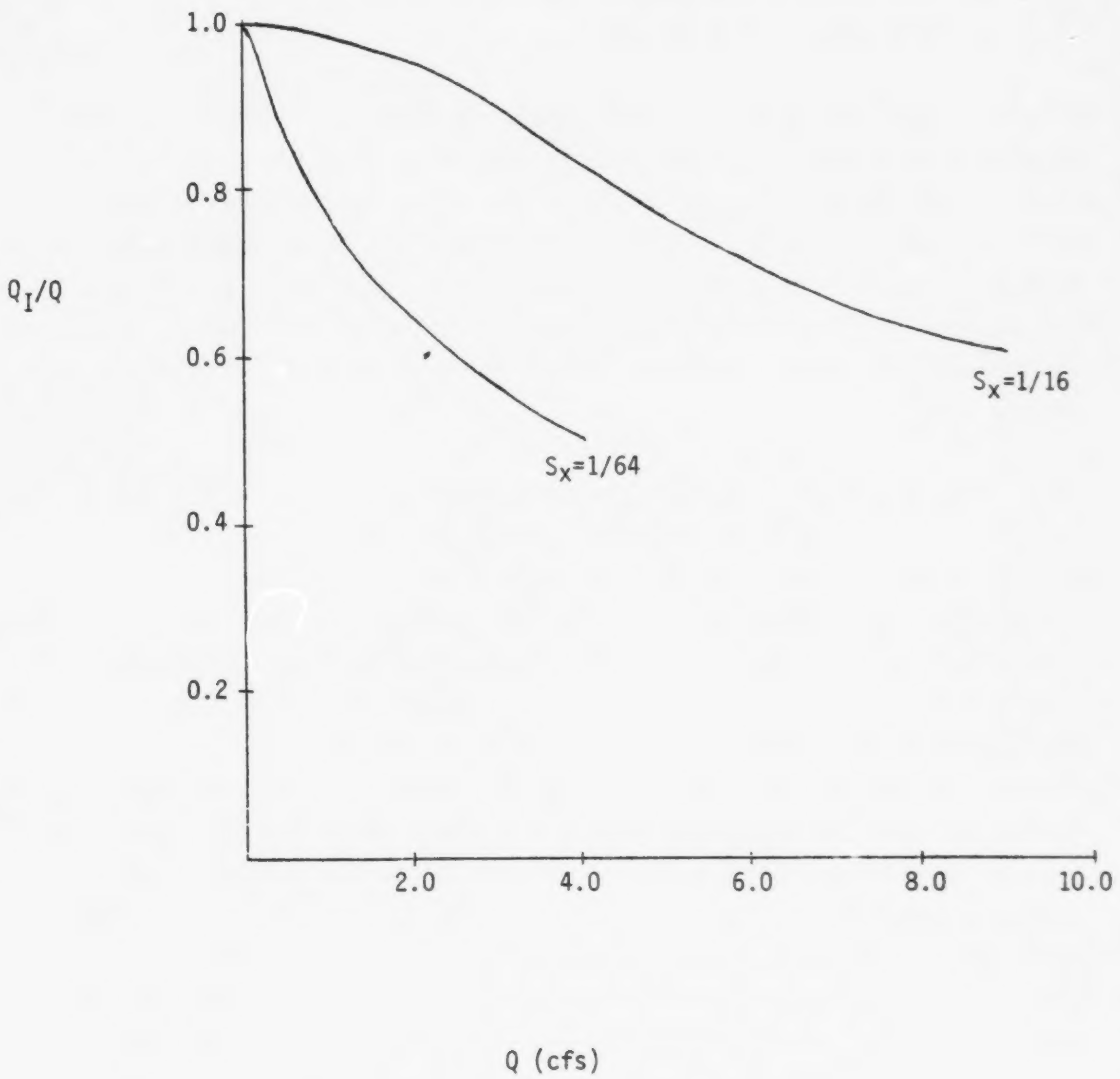

FIGURE 1

INLET EFFICIENCY CURVES FOR 2' X 4'
PARALLEL BAR GRATE INLET WITH GUTTER
SLOPE = 0.02 (7)

channel flow, determines the inlet efficiency from the curve, and computes the flow intercepted by the inlet and the flow carried over to the next gutter/channel section.

Kinematic Cascades for Overland Flow

In order to account for the peculiar geometry and surface types of the drainage area associated with the highway right-of-way, it was decided to modify the overland flow routines used in the Runoff Block of SWMM. Basically, SWMM employs the kinematic wave approximation for overland flow, accounting for infiltration by Horton's equation and for depression storage (8). Each catchment is characterized by a drainage area, width, slope, roughness coefficient, infiltration type and depression storage and drains directly to a surface gutter or channel. This approach has been modified in the Inlet Design Program and the Surface Runoff Program to allow runoff to cascade over as many as three subcatchments for each catchment before reaching a surface gutter or channel. Each of the three subcatchments can thus have a unique set of hydrogeometric characteristics, as listed above (with the exception of the catchment width which must be the same for each subcatchment).

The basic computational difference caused by addition of this kinematic cascade appears in the continuity equation for overland flow. Figure 2 summarizes the basic flow computations for a given time step for a typical subcatchment. In Figure 2, d_0 is the depth of flow at the previous time step, d_1 is the depth of flow at the current time step, and d_s is the maximum depth of depression storage. The continuity or storage equation is then given by:

$$\frac{\Delta d}{\Delta t} = R - I + \frac{(Q_i - Q)}{A_s}$$

where

$\Delta d = d_1 - d_0$;

R = rainfall intensity during the time step;

I = infiltration rate during the time step;

Q = outflow from the subcatchment;

Q_i = inflow from the upstream subcatchment; and

A_s = surface area of the subcatchment.

Thus, the basic difference is the addition of the inflow term Q_i in this equation.

The remainder of the solution for overland flow proceeds in basically the same manner as in SWMM. Infiltration, I, is computed with

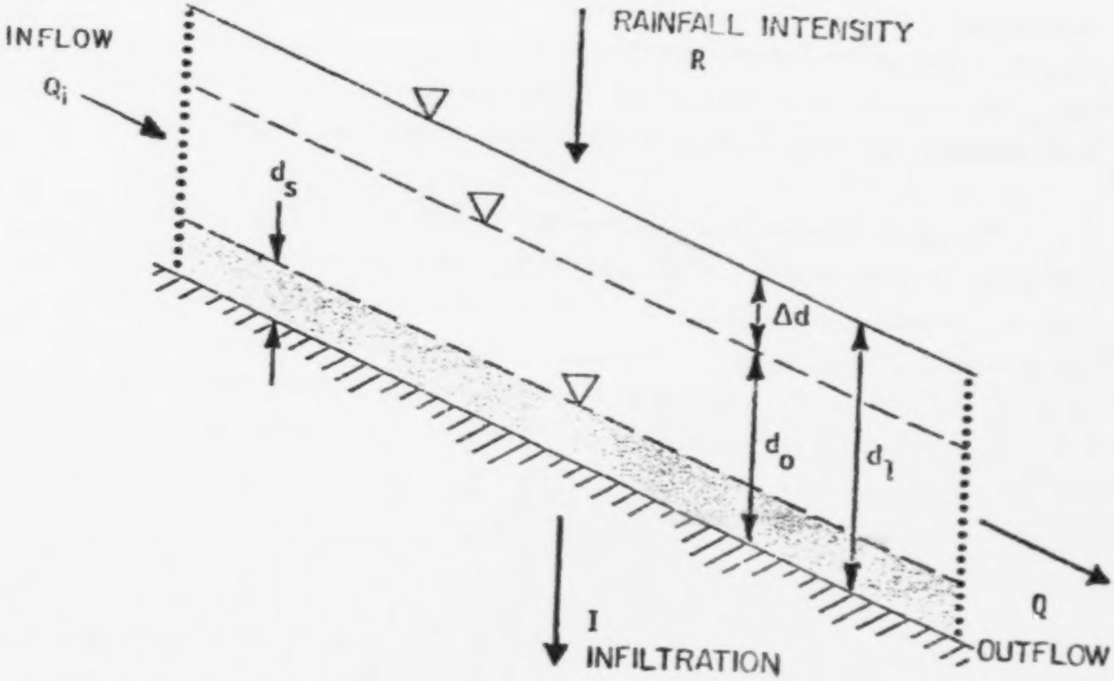
INFLOW
Q_i
RAINFALL INTENSITY
R
d_s
Δd
d_o
d_l
Q
OUTFLOW
I
INFILTRATION

FIGURE 2

RAINFALL-RUNOFF FROM A
TYPICAL SUBCATCHMENT

Horton's equation; outflow Q is given by the Manning equation (wide channel assumption). The continuity equation and the outflow equation are combined and solved by the Newton-Raphson iterative technique.

SUMMARY

This paper has presented an overview of an Urban Highway Storm Drainage Model developed for the Federal Highway Administration. The Model consists of a powerful and flexible set of computer programs for highway drainage analysis and design. The capabilities of the Model include statistical analysis of rainfall records, design of drainage systems including locating inlets and sizing pipes, analysis of drainage systems under extreme storm events, and simulation of runoff quality from the highway corridor.

Some of the computer programs described in this paper employ the same basic solutions for rainfall-runoff and flow routing as the EPA SWMM package; however, there are some major differences. Two significant modifications to the SWMM approach have been described in this paper - the inclusion of the simulation of inlet hydraulics and the addition of kinematic cascades for overland flow.

The Model is presently being tested on five typical highway sites around the United States. The user's manuals and documentation reports for the Model are being completed and should be available in the near future from the Federal Highway Administration.

ACKNOWLEDGEMENTS

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KINEMATIC DESIGN STORMS INCORPORATING SPATIAL AND TIME AVERAGING

by

W. James and J.J. Drake*

ABSTRACT

The paper explores the use of a numerical storm model as a pre-processor for a detailed urban runoff model. The proposed storm model generates hyetographs for each subcatchment, thus simulating the spatial and temporal growth and decay of a system of storm cells as they move across an urban catchment system.

Traditionally, design storms were and continued to be developed from statistical analysis of point rainfall records that include all types of rainstorms; the methodology was originally appropriate for flood predictions based on the rational formula. The resultant rain distributions are unlike any type of observed rain storm. This synthetic temporal distribution is typically applied uniformly across the catchment and flow hydrographs are consequently also unlike observed runoff hydrographs.

Large static cells of uniform rainfall intensity are rare even in prolonged frontal events. Convective cells tend to be circular with a circular rainfall intensity pattern. Rain cells tend to be elliptical, aligned sub-parallel to the front and moving sub-parallel to it. Rainfall is typically most intense near the leading edge of the cell. Fast moving storms produce very rapid point-intensity-duration changes. Statistics of the size and distribution of rain cells can be obtained most readily from weather radar studies. The storm model presented

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in this study is based on the synoptic observations of rain cells reported in weather radar literature.

Intensity-duration-frequency curves obtained from the model are similar to those derived statistically from the long-term rainfall record. The model is applied to catchments in the Hamilton area in Southern Ontario. Results indicate that storm cell kinematics are significant, particularly in peak runoff estimates (for drainage design) and water pollutant loading estimates, because of the sensitivity of the time-to-peak and rate-of-rise of hydrographs and pollutographs. Such storm models appear to be useful for computer-based rainfall-runoff studies.

INTRODUCTION

Modern rainfall-runoff models for use in urban hydrology, such as the Stormwater Management Model (Huber, 1975), allow up to six rainfall hyetographs, and discretization of the catchment into (typically) 40-100 subcatchments. With a little modification the programs can accept 10-20 hyetographs distributed across these subcatchments. It is then a simple matter to lag and adjust the hyetographs to represent a moving storm tracking across the catchment in any given direction, and also to account for probable growth and decay of the spatial size of the storm cell as well as change in the rainfall intensity distribution across the cells.

In fact it is unlikely that storms will spontaneously occur, grow and die off while remaining stationary over a typically small urban basin. It can be shown that translatory storms produce very different runoff hydrographs and pollutant loadings than those produced by the usually-accepted static storm. NOT to account for translatory storms may introduce unjustified errors.

DESIGN STORMS AS MODEL INPUT

Generally a prescribed return period is specified for a design project and then a design storm that has the same return period is selected. The project is designed not to fail when subjected to a calculated flood produced by the design storm; it is assumed that the capacity of the design drainage system has a return period equal to that of the design storm.

The validity of this assumption of a linear relationship between some measure of the runoff hydrograph and a description of the rainfall hyetograph is of importance (Wenzel and Voorhees, 1978 and 1979) and open to question. This is especially true where antecedent moisture conditions are significant and variable and where surcharging is involved. Continuous modelling, though expensive, is seen to be helpful in this regard, but few continuous models account for the complex actions of the combined sewer overflow and diversion structures commonly encountered.

Design storms are usually either developed by a simple statistical analysis of point rainfall records that include rain of all types, or an historic storm is used (particularly for rare events). In the former case the resulting temporal distribution of rain may be quite unlike the rainstorms occuring in nature (Urbonas, 1979) and thus result in runoff peaks that can vary significantly from peak flows calculated statistically from long-term simulation using recorded rainfall and calibrated models. The use of storm models based on known synoptic characteristics of storms does not seem to have been considered a design alternative. Synthetic storms attempt to aggregate intensity duration stations from many storms of all types, thunderstorms and cyclonic rains, into a single and hence impossible storm event. The general design hyetograph shape is based on data from many geographic regions, some involving orographic and other local effects.

A number of studies have been carried out on the various published methods of synthesizing design storms from rainfall records (see for example, Marsalek, 1978; Arnell 1978) and these draw conclusions regarding the adequacy or otherwise of the various synthetic design storm techniques. None of the studies available to us has accounted for the storm kinematics or dynamics, notwithstanding the fact that storm movement may significantly affect the computed catchment response, especially pollutographs, or pollutant loadings to the receiving waters. Most synthetic design storms are tantamont to an attempt to replace the several precipitation types and kinematics by a single simplistic rain hyetograph applied uniformly across a catchment, supposedly appropriate to all shapes, sizes and kinds of catchment.

A promising path through this jungle has been proposed by Walesh et al. (1979): hyetographs of major rainfall events are assembled from a long historic record and applied to a calibrated event model. Rain intensities thus still relate to actual precipitation types.Unfortunately, once again the historic record is usually based on independent single point rainfall observations, incapable of accounting for storm dynamics. Dahlstrom (1978) has suggested that the spatial variability of precipitation intensity can be taken into account by a complicated analysis of precipitation records from a limited number of adjacent stations for urban areas of large size. However, Dahlstrom evidently does not believe that current knowledge of meso-scale rain storm characteristics would justify the development of a useful storm model for urban hydrology. Both Dalstrom and Arnell appear to support Walesh's general approach to the use of many storms from the long-term rainfall record, but do not caution against overlooking storm kinematics.

A recent paper (Yen and Chow, 1980), although another attempt to apply a simple, approximate, spatially uniform, and static design hyetograph, does discuss the effect of the inherent storage and attenuation of a basin on the selection of the design hyetograph. These authors define a small basin as one sensitive to high-intensity rainfalls of short durations and sensitive to land use; inherent storage-channel characteristics do not suppress these sensitivities.

The problem is not new. Clark (1973) has emphasized the case for including inherent stochasticity and error in the modelling procedure and has discussed this relationship to the observation network density. Van Nguyen et al. (1973) advocate the use of a radar system rather than a dense network of rain gauges. Both of these studies appear to hint at the need to model moving storms.

In summary, the trend seems to be increasingly critical of the use of a simple static, spatially-uniform design storm aggregating all kinds of storm types, and towards the use of a number of historic storms selected from the long-term record, or even continuous modelling using the entire long-term record. There is also increasing interest in better sampling of the rainfall inputs, but at present the accent is on data analysis rather than modelling of dynamic storms. Hydrometeorologists have perhaps been too cautious to advocate the use of storm models incorporating cell kinematics.

PRECIPITATION TYPES SIGNIFICANT TO STORMWATER MODELLING

Three types of rainfall are generally recognized: orographic, cyclonic and convective. In the Hamilton area, orographic uplift over the Niagara escarpment rarely leads to rainfall although it may generate clouds. Cyclonic precipitation, associated with frontal systems, has two components: a broad belt of relatively low intensity rainfall lying along the warm front, and a narrower belt of relatively high intensity rainfall lying along the following cold front. Convective precipitation in summer may be caused by differential local surface heating and generate small intense rain cells, or the cold front rainband may in fact be composed of a linear set of convective cells associated with the air mass moving from Lake Erie over the land. Occasionally, very intense, widespread convective rainfall may be associated with the incursion of a tropical storm into the area (e.g. Hurricane Hazel in 1954).

Adiabatic cooling is the cause of condensation and rainfall, and vertical transport of humid air masses is a requirement. In convective precipitation, heated air at the ground expands, reduces in weight, and takes up increasing quantities of water vapour. The warm moisture-laden air becomes unstable and pronounced vertical currents are developed. Dynamic cooling then causes condensation and precipitation in the form of light showers, storms or thunderstorms of high intensity.

Thunderstorms begin as cumulus clouds characterized by strong updrafts that reach 25,000 feet. During development of the storm additional moisture is provided by a considerable horizontal inflow of air. The storm enters a mature stage when the strong updrafts produce precipitation. Gusty surface winds move outward from the region of rainfall and heavy rainfall occurs for a period of 15 to 30 minutes. In the final dissipating stage of the storm the downdrafts predominate and precipitation tails off and ends.

Thunderstorms comprise one or more such cells in varying stages of development, the life cycle of which is usually completed in an hour or less. However, such storms tend to be self-propagating by the formation of new cells and in the Hamilton area generally move from the west (between, say, north west and south west) at speeds of 35-50 km/hr and in broken lines or bands up to 80 km in width. Severe storms may produce 5 cm of rain in less than half an hour, while slowly moving storms may appear to remain in one locality for an hour or more and produce a total point rainfall as great as 30 cm.

In the Hamilton urban catchment the greatest rainfall rates are associated with convective precipitation. Although major structures may be designed on the basis of an exceptional recorded event, urban stormwater structures are designed on the basis of a composite synthetic storm, derived from raingauge data that is assumed to begin, peak and end simultaneously over the whole catchment. In fact, the greatest rates of runoff are usually associated with a linear set of convection cells containing individuals that are continually being generated and dissipated, and which moves across the area.

In summary, rain storms travel in preferred directions; they do not spontaneously grow and die over one spot, as suggested by current practice in the analysis of point rainfall data. Cells have substantial speeds and intensity variations across areas typically appropriate to urban runoff studies (e.g. 5-5000 acres). A substantial body of information is available and the general characteristics of stormcells can be described.

It is preferable to specify the expected speed and direction of movement of cells, and even cell size and rainfall intensity distribution, rather than to assume no speed or direction, and excessively large cells with uniform rainfall intensities.

Finally, thunderstorms have different characteristics from cyclonic events. Point rainfall data does not distinguish between rainfall types, and statistical analyses of rain data includes intensities from all types. Point rainfall data cannot generally provide information on storm cell kinematics.

THE STORM MODEL

Radar studies of summer rain events resulting from moving clusters of sub-circular convective cells have shown that the cells are relatively short-lived (Austin 1967), that they tend to have an exponentially-decreasing intensity away from the cell centre (Konrad, 1978) and that their statistical properties can be matched to those of ground-based precipitation records (Drufaca, 1977). Gupta and Waymire (1979) have proposed a stochastic model of rainfall from such clusters, but no *a priori* single-event design storm for the Hamilton area can be derived from it.

Studies of "line-convection" rainbands associated with extra-tropical cyclones have shown them to be longer lasting and their structure to be one of sets of extended elliptical cells oriented sub-parallel to the front with a component of motion along it (Hobbs and Brewer, 1979; James and Browning, 1979). This pattern of rainfall, oriented across the drainage basin and moving down from head to mouth is apparently common in Hamilton, and forms the basis of the present model. The form of the model is an infinitely wide rainband in which the rainfall intensity decays exponentially away from the line of peak intensity at different rates ahead and behind it. Thus

$$P = P_o e^{-k_1(t_p - t)} \qquad t < t_p$$

$$P = P_o e^{-k_2(t - t_p)} \qquad t > t_p$$

where P_o is the instantaneous point peak intensity t_p is the time-of-peak at a point and t is time at a point. Statistical studies of rainfall rates before and after peak rates recorded by raingauges in several parts of North America indicate that $k_2 = 0.54\ k_1$ and we have adopted this value. There are therefore two parameters of the model, P and k_1 (or k_2), which can be evaluated from intensity-frequency-duration curves published for the Hamilton area by assuming that events of all durations for a given recurrence interval are embedded in one storm. Figure 1 shows that the model i-f-d curves for which the parameters have been evaluated with the 10 and 20 minute points from the observed curves provide a close approximation to those curves. Projected instantaneous point maximum intensities (P_o), and the values of k_1 and k_2 are shown in Table 1. Konrad (1978) and other studies have suggested that convective cells with similar peak intensities have an intensity distance-decay exponent of about 0.5 km^{-1}. If this value is assumed to be correct for linear rainbands in the Hamilton area storm velocities can be calculated, and these are also shown in Table 1

Table 1

Return period yr	P_o mm hr^{-1}	k_1 mm^{-1}	k_2 mm^{-1}	v km hr^{-1}
10	157	0.19	0.10	15
50	188	0.15	0.08	13
100	196	0.13	0.07	11

The calculated values are similar to those observed for motion of linear rainbands normal to their orientation on the radar studies. It appears that increased rainfall for a given duration at longer return periods is due to both an increased intensity in the storm and to a slower motion of it across the area in question.

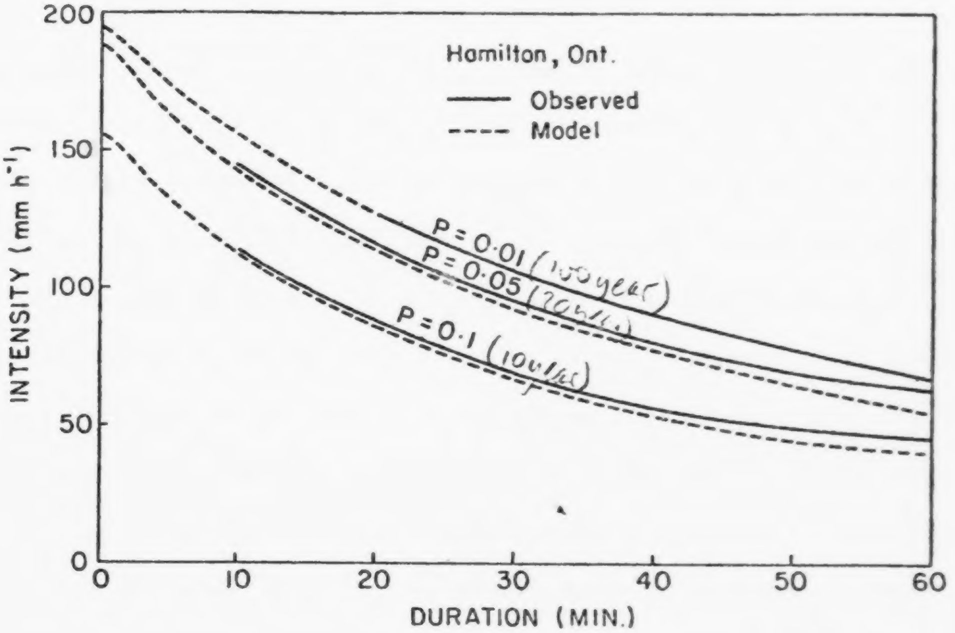

Figure 1 Intensity-Duration-Frequency Curves Obtained by the Storm Model

APPLICATION OF THE MODEL

One a model of a rain cell is proposed it is easy to show the effect of the necessary time and spatial-averaging in order to produce expected runoff hydrographs. Fig. 2 shows the instantaneous cross-section of a typical line convection cell, having a peak intensity of 180 mm/hr.

The effect of storm kinematics is shown in Figure 3. The greater the cell speed across the raingauge, the sharper the hyetograph. This is equivalent to the point data recorded by rain gauges and used in design storm determination. Hence the statistics are significantly affected by storm velocities.

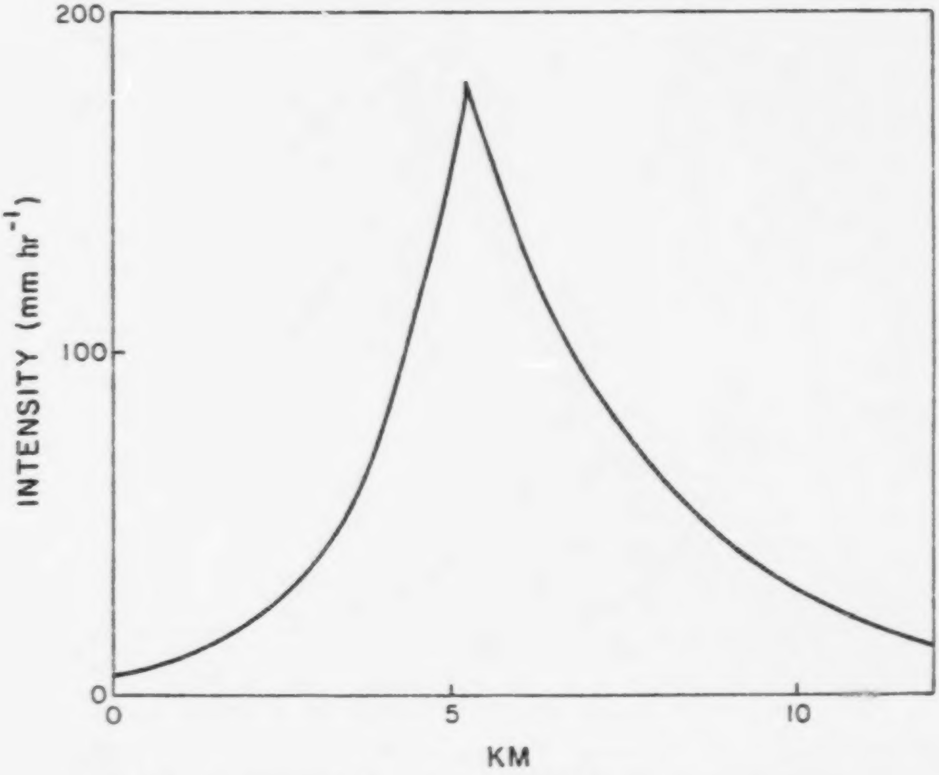

Fig. 2. Cross-section of typical line convection cell

In order to model runoff it is necessary to produce input hyetographs averaged over each subcathment or spatially discretized area, and also over the concomitant basic computational time-step. The effects are shown respectivly in Figures 4 and 5.

Typical hydrographs for the Chedoke Creek catchment in Hamilton are shown in Figures 6 and 7 for the rainfall inputs shown in Figure 3.

CONCLUSIONS

Ground-based estimates of areal rainfall show considerably less attenuation from the maximum point value than do radar-based estimates (e.g.

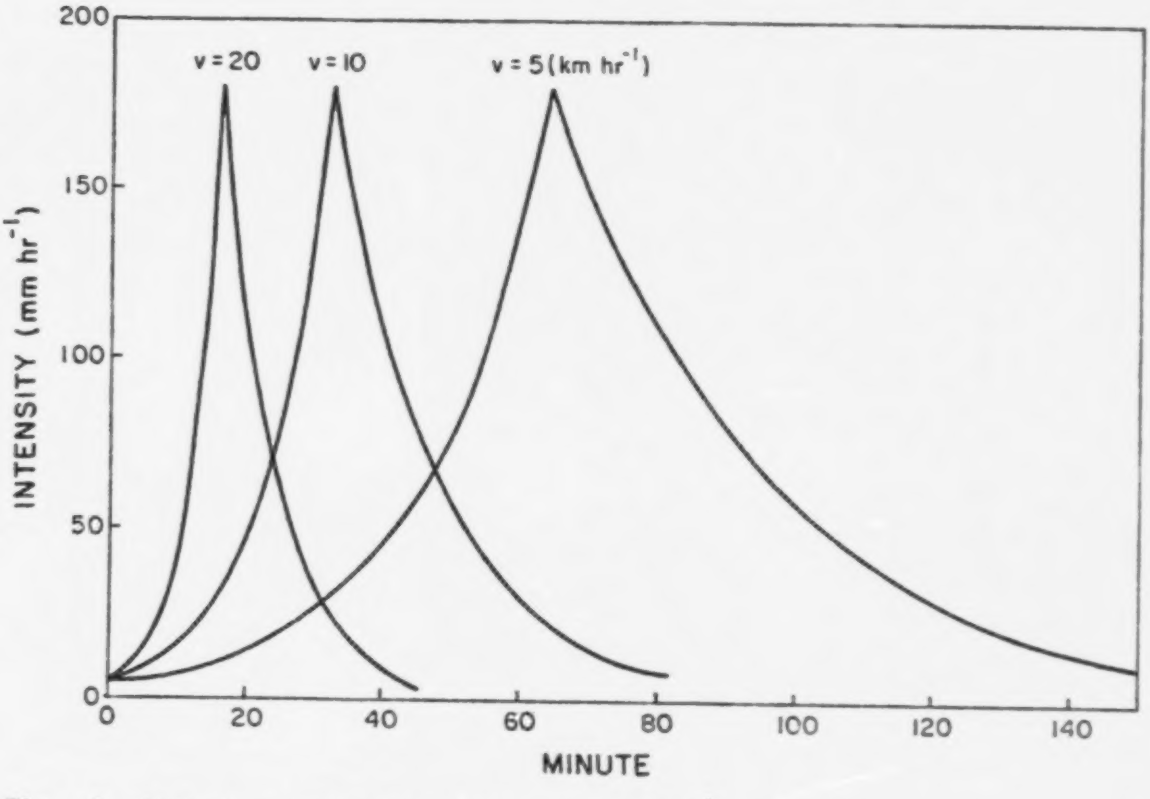

Fig. 3. Effect of velocity (5, 10, 20 km hr^{-1}) on time-distribution of instantaneous rain intensity from storm shown in Fig.2

Barge et al, 1979), because they do not take into account the translation of a storm across the area. The implication of these studies for operational urban hydrology is thus that a given duration-frequency event in a given catchment may produce a smaller total volume of rain than is currently assumed from depth-area curves based on raingauge records, but that this volume may be distributed in space and time such that it gives rise to a greater peak runoff and rate-of-rise of the hydrograph.

Thus preliminary analysis has shown that the effects of the kinematics of storms on urban stormwater hydrographs and pollutographs can be significant to both modelling and design. More detailed information on the structure and

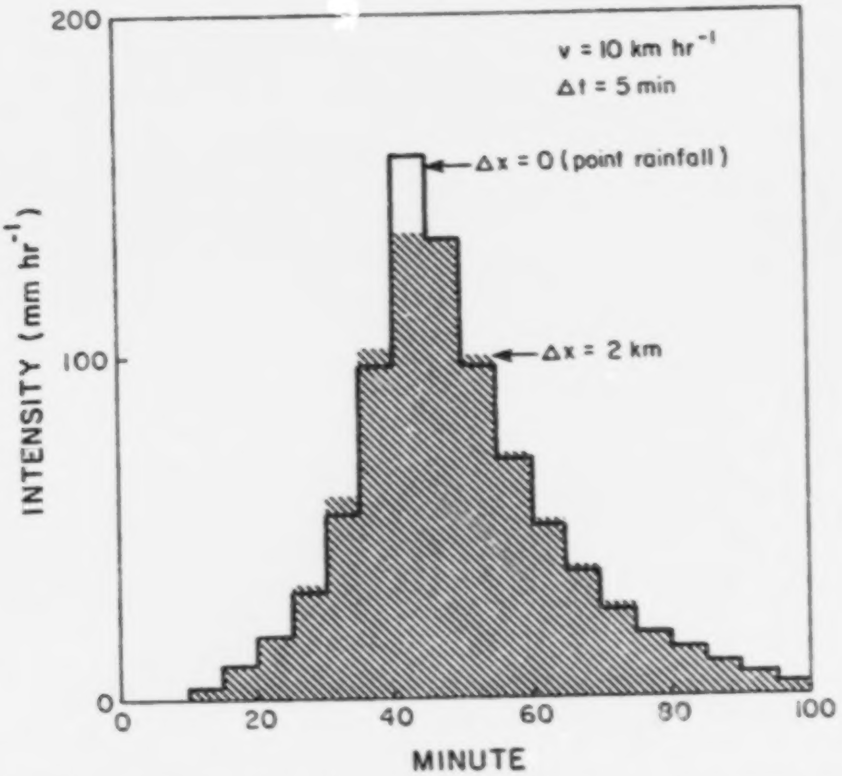

Fig. 4. Effect of basin size (0, 2 km length) on 5 minute rain intensities from storm shown in Fig. 3. moving across basin at 10 km hr^{-1}

kinematics of rainfall in the Hamilton urban catchment can be derived from data from the A.E.S. weather radar that has operated for the past 10 years at Woodbridge, Ontario.

ACKNOWLEDGEMENTS

The work reported here is part of a joint study on the Chedoke Creek catchment. Assistance from the Regional Engineering Department of the Hamilton-Wentworth Regional Municipality is gratefully acknowledged.

The runoff computations for the Chedoke catchment were carried out by Zvi Shtifter, graduate student in the Department of Civil Engineering at McMaster University.

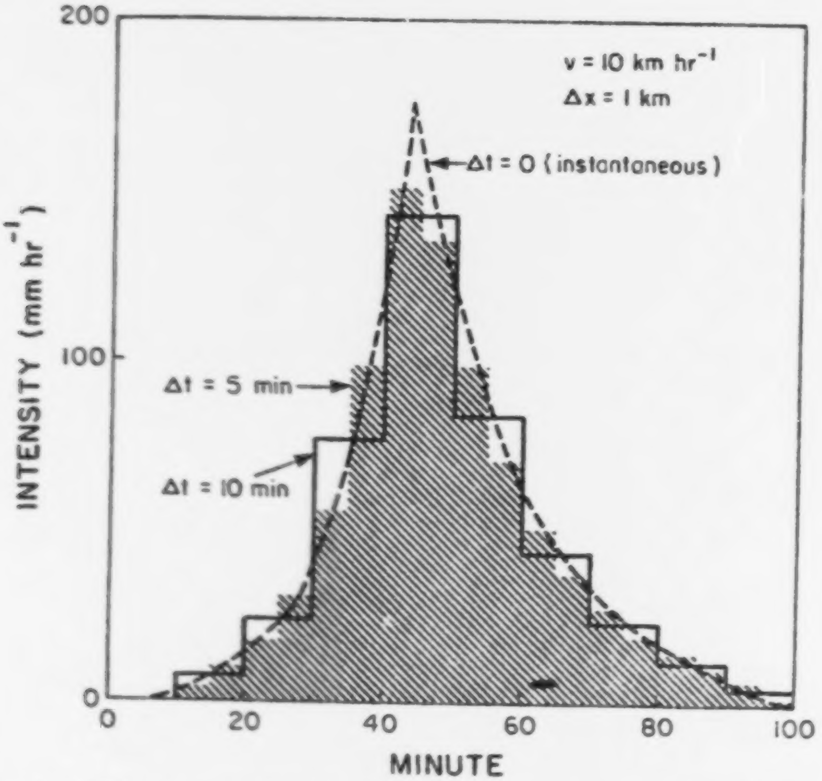

Fig. 5. Effect of time averaging (0,5,10 min) on average rain intensities over a 1 km long basin resulting from storm shown in Fig. 3 moving across basin at 10 km hr.

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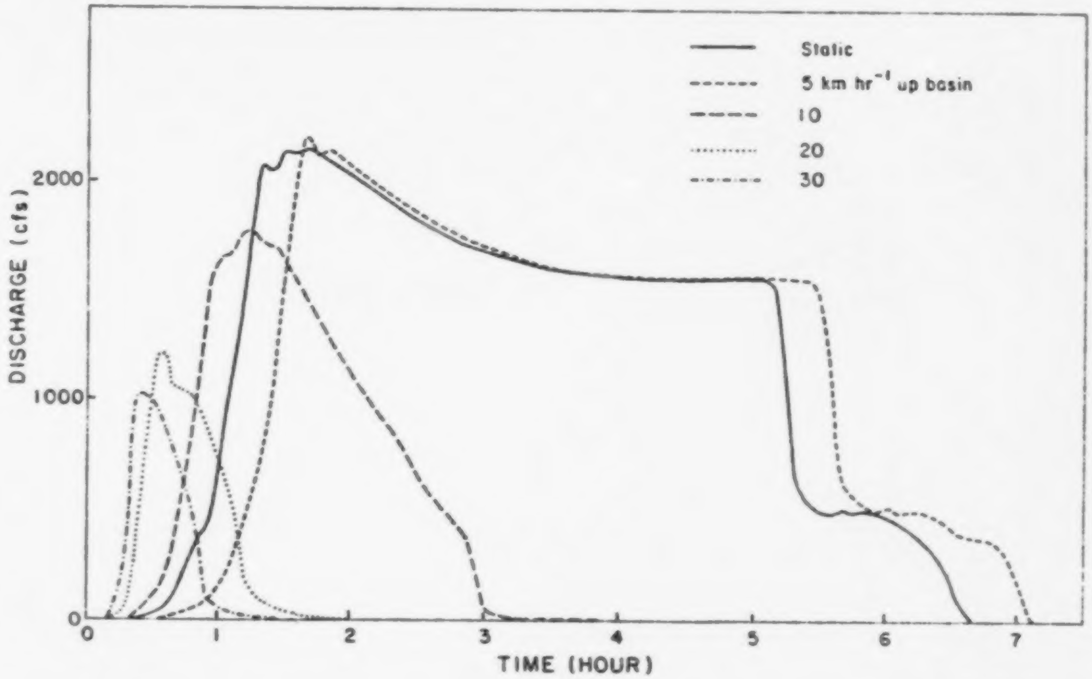

Fig. 6 Computed runoff for a storm moving up the Chedoke Creek catchment

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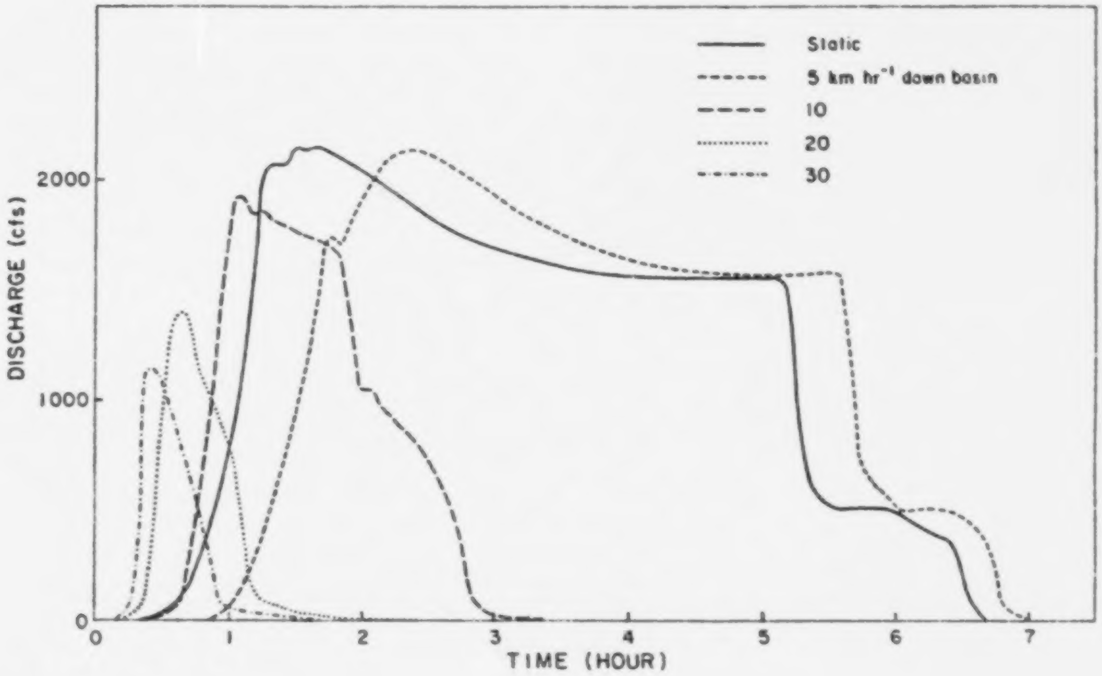

Fig. 7 Computed runoff for a storm moving down the Chedoke Creek catchment.

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HYDROGRAPH SYNTHESIS
BY THE HNV-SBUH METHOD
UTILIZING A PROGRAMMABLE CALCULATOR

by

Bernard L. Golding[1], P. E., P. Eng.

Introduction:

The determination of the complete flow hydrograph for retention/detention basin design is now a prerequisite to the formulation of storm water management plans for most new developments. They are also generally required for the investigation of existing urban drainage basins. At the present time, most engineers use the SCS procedure (the computation of rainfall-excess by the SCS rainfall-runoff equation and the application of the computed rainfall-excess increments to the standard SCS unit hydrograph) for hydrograph computation because of its simplicity and general low cost even though its deficiencies have now been recognized by many engineers. The alternative is the use of more sophisticated models (i.e., SWMM, ILLUDAS, HEC-1, etc.) whose fairly high costs and complexity are not always warranted considering the benefits achieved.

The purpose of this paper is to acquaint engineers with the Howard Needles Version of the Santa Barbara Urban Hydrograph Method (HNV-SBUH Method), a fairly simple, yet easily applied low cost model for computing design flow hydrographs. Included with this paper are calculator programs and user instructions for developing hydrographs by this model utilizing the Hewlett-Packard HP-97 and HP-67 programmable pocket calculators. A BASIC language version of these programs for microcomputers and user instructions for their application is also available. The validity of the model by the simulation of actual runoff events utilizing the programs listed herein is also demonstrated.

General:

The HNV-Santa Barbara Urban Hydrograph Method (HNV-SBUH Method) is a modification of a method originally developed by Mr. James M. Stubchaer, F. ASCE, of the Santa Barbara County (California) Flood Control and Water

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[1] Chief, Water Resources (South), Howard Needles Tammen & Bergendoff, Orlando, Florida

Conservation District (1). The method as described herein computes a hydrograph directly without going through any intermediate process, as does the unit hydrograph method.

The HNV-SBUH Method is in many respects similar to some of the time-area-concentration curve procedures for hydrograph computation in which an instantaneous hydrograph in a basin is developed and then routed through an element of linear storage to determine basin response. However, in the HNV-SBUH Method, the final design (outflow) hydrograph is obtained by routing the instantaneous hydrograph for each time period (obtained by multiplying the various incremental rainfall excesses by the entire watershed area in acres) through an imaginary linear reservoir with a routing constant equivalent to the time of concentration of the drainage basin. Therefore, the difficult and time consuming process of preparing a time-area-concentration curve for the basin is eliminated.

Model Description:

A step-by-step description of the basic HNV-SBUH Method is given below:

1. Runoff depths for each time period are calculated using the following equations:

(1) Directly Connected Impervious Area Runoff -
$R(0) = I \cdot P(t)$ (inches) (1)

(2) Pervious Area Runoff -
$R(1) = P(t)\,(1-I)-f(1-I_t)$ (inches) (2)

(3) Total Runoff Depth -
$(R(t) = R(0) + R(1)$ (inches) (3)

where

$P(t)$ = Rainfall depth during time increment Δt (inches)
f = Infiltration during time increment Δt (inches)
I_t = Total impervious portion of drainage basin (decimal)
I = Directly connected impervious portion of basin (decimal)
Δt = Incremental time period (hours, i.e., 0.25, 0.50, etc.)

The directly connected impervious areas (I or DCIA), sometimes referred to in the literature as the hydraulically effective impervious area, are those impervious areas where runoff therefrom does not flow over a pervious area before reaching and entering an element of the drainage system (streets with curbs, catch basins, storm drains, etc.).

In the derivation of Equation 2 above and as shown on Figure 1, the rainfall on the pervious area is considered to be made up of two parts, the rainfall on the nondirectly connected impervious area $(I_t-I)P(t)$ which is assumed to run off uniformly onto and to be distributed uniformly over the pervious area and the rainfall on the pervious area $(1-I_t)P(t)$ which, when added together, gives the equation $(1-I)P(t)$. The runoff from the pervious area is then obtained by subtracting infiltration $f\ (1-I_t)$ from this total rainfall or $P(t)(1-I)-f(1-I_t)$.

2. The instantaneous hydrograph is then computed by multiplying the total runoff depth $R(t)$ for each time period t by the drainage basin area A in acres and dividing by the time increment Δt in hours.

$$(4) \quad I(t) = R(t) \quad A/\Delta t \qquad \text{(cfs)}$$

As in the Rational Method, the conversion factor 1.008 was ignored.

3. The final design (outflow) hydrograph $Q(t)$ is then obtained by routing the instantaneous hydrograph $I(t)$ through an imaginary reservoir with a time delay equal to the time of concentration (T_c) of the drainage basin. This flood routing may be done by use of the following equation which is subsequently derived on Table 1.

$$(5) \quad Q(t)=Q(t-1)+\bar{K}[1(t-1(+1(t)-20(t-1)] \quad \text{(cfs)}$$

where

$$K = \frac{\Delta t}{2T_c+\Delta t}$$

and where T_c = Time of concentration of the basin (hours).

In the HNV-SBUH Method, all of the rain which falls on the basin is considered runoff except for the first 0.1 inch (depression storage) which is automatically subtracted from the rainfall in the programs.

Infiltration:

In the programs for the HNV-SBUH Method subsequently listed, infiltration is computed in accordance with the relationships illustrated on Figure 2 as first described by Holtan and developed and presented by Terstriep and Stall (2). In this methodology, it is assumed that an Initial (Maximum)

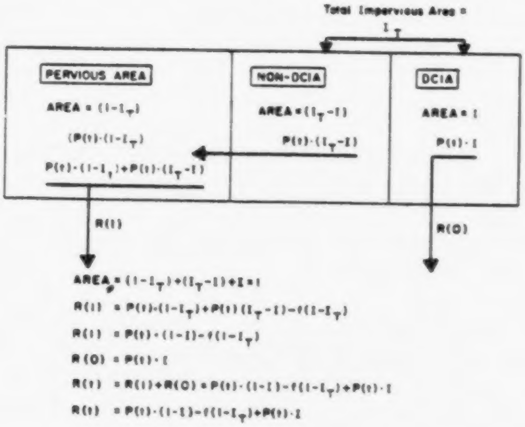
Total Impervious Area =
I_T
PERVIOUS AREA
AREA = $(1-I_T)$
$(P(t)\cdot(1-I_T)$
$P(t)\cdot(1-I_T)+P(t)\cdot(I_T-I)$
NON-DCIA
AREA = (I_T-I)
$P(t)\cdot(I_T-I)$
DCIA
AREA = I
$P(t)\cdot I$
R(I)
R(O)
$AREA_T = (1-I_T)+(I_T-I)+I = 1$
$R(I) = P(t)\cdot(1-I_T)+P(t)(I_T-I)-f(1-I_T)$
$R(I) = P(t)\cdot(1-I)-f(1-I_T)$
$R(O) = P(t)\cdot I$
$R(t) = R(I)+R(O) = P(t)\cdot(1-I)-f(1-I_T)+P(t)\cdot I$
$R(t) = P(t)\cdot(1-I)-f(1-I_T)+P(t)\cdot I$

FIGURE 1

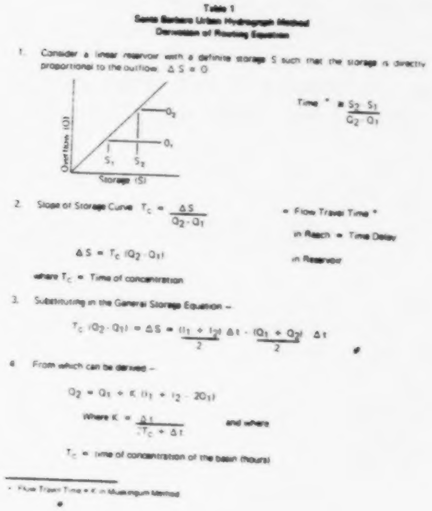

Table 1

Santa Barbara Urban Hydrograph Method

Derivation of Routing Equation

1. Consider a linear reservoir with a definite storage S such that the storage is directly proportional to the outflow. $\Delta S = Q$

Outflow (Q)
Storage (S)
Q_2
Q_1
S_1
S_2

$\text{Time}^* = \frac{S_2 - S_1}{Q_2 - Q_1}$

2. Slope of Storage Curve $T_c = \frac{\Delta S}{Q_2 - Q_1}$ = Flow Travel Time * in Reach = Time Delay in Reservoir

$\Delta S = T_c (Q_2 - Q_1)$

where T_c = Time of concentration

3. Substituting in the General Storage Equation –

$$T_c (Q_2 - Q_1) = \Delta S = \frac{(I_1 + I_2)}{2} \Delta t - \frac{(Q_1 + Q_2)}{2} \Delta t$$

4. From which can be derived –

$$Q_2 = Q_1 + K (I_1 + I_2 - 2Q_1)$$

Where $K = \frac{\Delta t}{2T_c + \Delta t}$ and where

T_c = time of concentration of the basin (hours)

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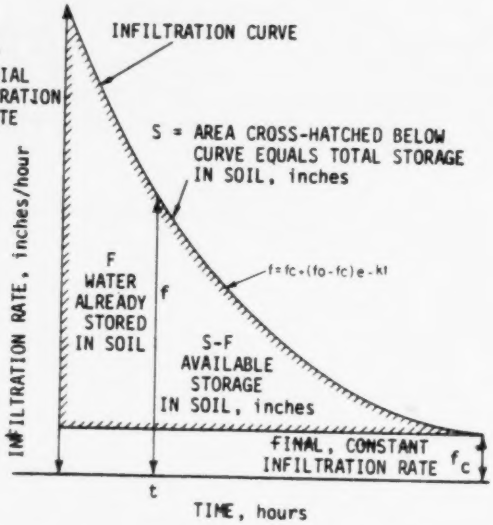
* Flow Travel Time = K in Muskingum Method



f_o
INITIAL INFILTRATION RATE
INFILTRATION CURVE
S = AREA CROSS-HATCHED BELOW CURVE EQUALS TOTAL STORAGE IN SOIL, inches
F
WATER ALREADY STORED IN SOIL
f
$f = fc + (fo - fc)e - kt$
S-F
AVAILABLE STORAGE IN SOIL, inches
FINAL, CONSTANT INFILTRATION RATE
f_c
INFILTRATION RATE, inches/hour
t
TIME, hours

Figure [illegible] Diagram of infiltration curve and infiltration rates as related to storage in soil

Infiltration Rate f_o in inches per hour is available in the soil mantle when rainfall starts which drops off exponentially with time as the voids in the soil become filled with water to some Final (Constant) Infiltration Rate f_c – also in inches per hour. The total cross-hatched area below the Infiltration Curve and above the Final (Constant) Infiltration Rate line (=S) represents the total available water storage in the soil mantle. As shown on Figure 2, this Total Available Water Storage Capacity, S, in the soil in inches is divided into two parts: 1) the water already stored in the soil when rainfall begins (F) which is the result of previous rainfalls, and 2) the actual Available Storage in the Soil (S-F). Of course, if a long dry period preceded the start of rainfall, the total area (actually a volume) under the Infiltration Curve (=S) would be available for water storage. In the computer programs the infiltration f shown on Figure 2 is computed by the Horton equation.

$$f-f_c+(f_o-f_c)e^{-kt}$$

where

f = infiltration rate at some time t after the start of rainfall, inches per hour
f_o = initial infiltration rate, inches per hour
f_c = final constant infiltration rate, inches per hour
k = a shape (decay) factor (=2)
e = base of natural logs

In using the HNV-SBUH Method, programs listed herein, the Standard Infiltration Curves for each of the four general U.S. Soil Conservation Service Groups (A, B, C and D) shown on Figure 3 as originally established by Terstriep and Stall and as modified by the writer (3) are used to compute infiltration. Standard Antecedent Moisture Conditions for these four types of soils, as also established by Terstriep and Stall and which are subsequently discussed, can be used to determine F. The various factors used to compute the Standard Infiltration Curves are shown on Figure 3 for each of the four SCS Standard Hydrologic Soil Groups are listed on Table 2.

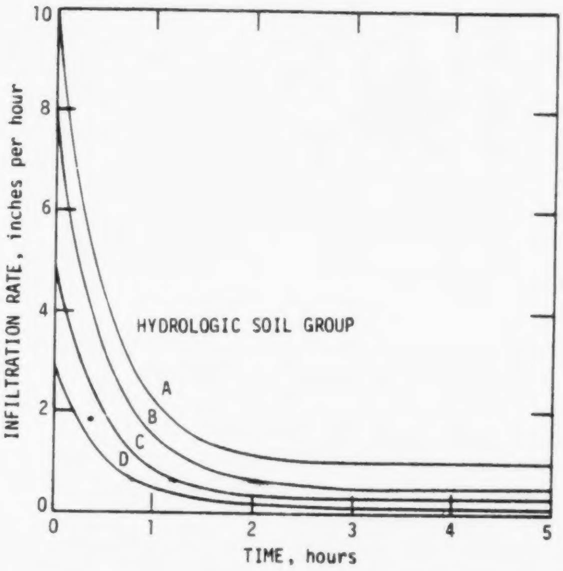


Figure 3 Standard infiltration curves for bluegrass turf

Table 2
Factors Used For Calculating The Standard Infiltration Curves For Grassed Areas

Item	Value			
Hydrologic soil group USDA designation	A	B	C	D
Final constant infiltration rate f_c, inches per hour	1.0	0.50	0.25	0.10
Initial infiltration rate, f_o, inches per hour	10	8	5	3
Shape factor, k, of infiltration curve	2	2	2	2
Available storage capacity, S, in soil mantle, inches, for four antecedent conditions				
Bone dry, condition 1	4.3	3.4	2.3	1.3
Rather dry, condition 2	3.4	2.8	1.8	1.1
Rather wet, condition 3	1.9	1.4	1.0	0.6
Saturated, condition 4	0	0	0	0
Infiltration accumulated, F, in soil mantle, inches, at start of rainfall				
Bone dry, condition 1	0	0	0	0
Rather dry, condition 2	1.9	1.4	1.0	0.6
Rather wet, condition 3	3.4	2.8	1.8	1.1
Saturated, condition 4	4.3	3.4	2.3	1.3

Table 3
Antecedent Moisture Conditions For Pervious Areas (grass)

Description	Total rainfall during 5 days preceding storm (inches)
Bone dry	0
Rather dry	0 to 0.5
Rather wet	0.5 to 1
Saturated	over 1

The four antecedent moisture conditions listed in this table - Bone Dry (Condition 1), Rather Dry (Condition 2), Rather Wet (Condition 3) and Saturated (Condition 4), are dependent on the total rainfall that occurred during the five days preceding the particular storm (Antecedent Moisture Condition) as shown in Table 3.

In the computer programs subsequently described, the infiltration f, at some particular value of F, is first computed for the particular Antecedent Moisture Condition by first summing up incremental volumes under the curve for increments of 0.01 hour, computing a second value of infiltration f_2 at incremental time Δt later and then averaging the two computed infiltration values. To compute rainfall-excess from the pervious areas, this average infiltration over the Δt time period is then applied to that rainfall-excess increment during which time element a total rainfall of 0.1 inch has fallen (depression storage).

Theoretically, the 0.1 initial depression storage should be subtracted from the rainfall and the infiltration curve applied only at that subsequent time after rainfall has started, at which the total volume of rainfall and the total volume of infiltration storage under the infiltration curve are equal. However, in the programs subsequently listed the infiltraton curve is applied immediately after rainfall begins and the 0.1 depression storage has been satisfied. Therefore, a storm with small initial amounts of rainfall falling over an extended period of time will cause a premature reduction of the infiltration rate (a decay of the infiltration rate with time), which will result in higher runoff than would actually be the case. However, since the infiltration accumulated in the soil matter due to antecedent rainfall (=F) is very broadly defined and since the pervious area contribution is generally small, this induced error is normally not significant.

As an alternative to the four standard infiltration curves shown on Figure 3 and as defined on Tables 2 and 3, site specific infiltration data obtained by means of infiltration testing may be used. Necessary infiltration tests to determine the initial and final infiltration rates and decay factors are simple to perform. A recent Federal Highway Administration publication (4) describes standard procedures for infiltration testing. Antecedent moisture conditions may be determined by means of rainfall gages.

Required Model Input Data:

In addition to successive rainfall increments, either actual increments for runoff simulation or those from an arbitrary design storm for design hydrograph computation, required inputs to the model are as previously indicated the total drainage basin area (acres); the total impervious area (decimal); the directly connected impervious area (decimal); the time of concentration the basin (hours) and the required soils infiltration information - the initial infiltration rate f_o (inches/hour) at the start of the rainfall; the final infiltration rate f_c (inches/hour) and the water stored in the soil F (inches) at the start of the rainfall.

Although the total impervious areas of most drainage basins can, in most cases, be readily determined from aerial photographs, the determination of the directly connected impervious areas cannot. This determination is particularly difficult in the case of the roofs of buildings where in some cases all or parts thereof are directly connected by roof drains and in other cases (many times in the same block), runoff from roofs drains onto the grassed areas, either by roof drains or directly from the periphery of the roof. Therefore, a detailed field inspection of the buildings, driveways, etc. in the drainage basin and the marking of those directly connected impervious portions on the aerial photographs is generally required. For larger basins, a sampling technique must generally be used because of the cost involved. If streamflow records are available, the runoff depths from small storms on dry watersheds may be used to determine the directly connected impervious area, (ratio of runoff to precipitation), as almost all of the runoff from small storms will be from those areas. Where no actual data is available or obtainable such as where hydrographs for a new development must be computed, values based on similar type developments or literature values can be used. Recent studies by Miller (5) and Alley and Veenhuis (6) indicate that the directly connected impervious area constitutes approximately 50 percent of the total impervious area of single family residential areas (all lot sizes including trailer courts), 50 to 75 percent of multifamily residential areas and 75 to 90 percent of commercial areas.

The time of concentration of a basin, which by definition is the travel time of flow from the most remote basin to the point of interest, is calculated in the usual manner by summing up the initial (overland flow) and channel travel times. The channel travel times, both in roadside gutters or swales and in the downstream storm sewer systems or collector channels, are relatively easy to compute. A simple calculator program for travel time in storm sewer systems which computes the velocity of each pipe flowing full (=flowing 1/2 full), the flow travel time in each pipe and then sums up the flow travel times is available from the author. The determination of the initial travel time (overland flow time), which in many instances (particularly in the case of small basins) is larger than the channel travel times, is more difficult to compute. For instance, the pervious areas which generally constitute much of the flow paths of overland flow may not contribute at all during the early portion of a rainfall (where $f>P(t)$). However, based on the simulations so far undertaken by the writer and as undertaken by Stubchaer utilizing the original version of the SBUH Method, a nominal initial flow travel time of some sort is necessary input to the model. This initial flow travel time may be either computed utilizing one of the overland flow travel times (time of concentration) equations such as the equation of Ragan and Durer (7), or estimated. Generally, the writer has used 5 to 10 minutes as the initial flow travel time from the center of a residential lot to the roadside gutter or swale.

However, in the HNV-SBUH Method, as in the original SBUH Method as developed by Stuchaer, or in other methods where the time of concentration is a principal parameter, such as the Linearized Subhydrographs Model (8), the determination of the time of concentration is obviously the weakest link. This is particularly so where trapezoidal or rectangular shaped open channels (not pipes) are the principal conveyance systems. This is, of course, the direct result of the fact that the velocity of flow in open channels increases with depth which in turn reduces the travel time in such systems. However, the use of the velocity of flow at bank full stage to compute flow travel time appears to give a fair approximation of such travel time and has been used by the writer for design purposes.

Also, the HNV-SBUH Method makes the assumption that the flow in the channel is unrestricted by inadequate conveyance system capacity (i.e., inadequate pipe sizes). Where such inadequate system capacity exists, the time of concentration should be increased to reflect resultant street ponding (storage).

Model Validation

General: Four urban drainage basins of different sizes, locations and character were selected for use in model validation; hereinafter referred to as Basin Nos. 1 thru 4. All were well documented as to their area, physical properties, land use, soils types, etc., and all had excellent, accurate recorded rainfall-runoff information available including information on the amount of antecedent rainfall prior to the particular event simulated. The properties of each of the four basins used in the validation effort are listed on Table 4 and are described in detail elsewhere. (9) (10) (2)

TABLE 4

PROPERTIES OF DRAINAGE BASINS

Basin No.	Basin Name	Total Area (Acres)	Contributing Area (Acres)				SCS Hydrologic Soil Group	Time of Concentration (Minutes)
			Total	Imp.	DCIA	Pervious		
1	Multifamily Residential	14.7 *	14.7	10.4	6.5	4.3	D	15
2	Transportation (Highway)	58.3	26.4	15.8	10.5	10.6	A	20
3	Malvern Catchment	57.6	57.6	19.5	17.9	38.1	B[1]	15
4	Boneyard Creek	2380	1145	833	534	312	B	60

[1] Assumed to be in Group B; not classified by Marsalek.

Basin No. 1, the USGS's Multifamily Residential Area, in Dade County, Florida, 14.7 acres in size, consists principally of a very flat area essentially covered by apartment buildings and adjacent parking lots. Runoff from this site is collected by a Y-shaped (in plan) storm sewer system terminating in a 48" diameter pipe in which stage was monitored. The streets have no curbs or gutters; surface runoff from which is collected in the center of the streets which are depressed to act as a swale.

The soil of the pervious portion of the basin which constitutes 19.3 percent of the total area consists of Perrine Marl which has a very low infiltration rate (Hydrologic Soil Group D).

Basin No. 2, the USGS's Transportation (Highway) Site, in Broward County, Florida, 58.3 acres in size, consists essentially of a very flat 3,000 foot long segment of a six lane divided highway and adjacent contributary area. Runoff from this site is collected by a conventional circular reinforced concrete pipe storm drain system running on one side of the highway. The adjacent contributary area consists of a mixture of small commercial, residential and open, undeveloped areas. The base soil consists of fine sand (Hydrologic Soil Group A) such that runoff is generally less than 20 percent of the rainfall (from the 10.5 acres of directly connected impervious area). However, the time of concentration of the basin (=20 minutes) generally reflects 15 minutes of flow travel time in the storm sewer system and 5 minutes of flow overland travel time on a directly connected impervious area on its extreme end. Certain of the undeveloped portions of the basin are extremely flat and also extremely rough, overgrown with vegetation and contain significant depression storage and do not contribute runoff as will be subsequently described.

Basin No. 3, The Malvern Test Catchment in Burlington, Ontario, 57.6 acres in size, is a gently sloping (1%) single family residential area drained by a conventional storm sewer system. Input to the storm sewer system is by means of catch basins located on both sides of the curbed roadway system and in the swales which drain the backyards of the houses.

As described by Marsalek (10), 19.51 acres of this 57.60 acres basin is impervious of which area 17.88 acres are directly connected impervious area; the 1.63 acres of sidewalk area are not directly connected.

As also described by Marsalek the soils are sandy loam with an initial infiltration rate f_o of 3.0 inches/hour, a final infiltration rate of f_c of 0.52 inch per hour and a shape (decay) factor K of 4 day^{-1}

Basin No. 4, the USGS's Boneyard Creek Basin, located in Champaign-Urbana, Illinois, consists of a large essentially fully developed, fairly flat urban area 2,290 acres in size. This basin contains portions of the University of Illinois campus, old and new residential areas and a sizable commercial area. Runoff is collected by a large network of storm sewer systems which conduct flow into an open channel collector system approximately 3 miles in length. According to Terstriep and Stall (2), only 1,165 acres of this 2,290 acre basin actually contribute flow (for the intensity of the storms simulated) of which 830 acres are impervious and of which 534 acres are directly connected. The soils in the basin consist, again according to Terstriep and Stall, of silty loams (Hydrologic Soil Group B). For purposes of simulating runoff from this "reduced" basin an "average" time of concentration of 60 minutes was used rather than the 72 minutes as originally determined by Terstriep and Stall for the entire basin (11).

Close proximity enabled a visual inspection of the first two basins. From this visual inspection it was further concluded that only approximately 50% of the pervious area of Basin No. 2 (the USGS's Transportation Basin), and the nondirectly connected impervious area contributing thereto, contributed to runoff from the storm events simulated (0.5" to 2.0" of rainfall) because of the large depression storage inherent in the undeveloped, pervious portions of this basin which estimate, as will be subsequently described, resulted in the best simulation of actual events. From the visual inspection of Basin No. 1 (the USGS's Multifamily Residential Basin), it appeared that perhaps 5 to 10 percent of the pervious area and nondirectly connected impervious area contributary thereto might not contribute to runoff from this basin from the storm events simulated. However, the assumption was made that the entire basin contributed which resulted in satisfactory simulation of actual runoff events so that effect of a reduction in contributary drainage area was not investigated.

In the simulation of runoff from the first two basins, five minute duration rainfall increments were used: 1 minute, 2 minute and 10 minute increments for Basin No. 3; and six minute increments for Basin No. 4.

Basin No. 1: In all, runoff from eight rainfall events on the USGS's Multifamily Residential Area were simulated; the results of which simulations are shown on Figures 4 thru 11. The rainfall events simulated were essentially selected at random. However, care was taken to choose rainfall events with a reasonably wide range of antecedent rainfall conditions. For all events simulated, a time of concentration of 15 minutes was used in the simulations; 10 minutes computed pipe flow time assuming at an n=0.012 plus 5 minutes of overland flow time. The results of the simulations; the date of each storm event simulated; the antecedent moisture condition at the onset of the storm; the recorded and simulated peaks and volumes; and the percent errors are listed on Table 5. As can be observed from the figures and from Table 5, the simulations of runoff from Basin No. 1 were fairly good in all cases.

In order to determine the sensitivity of the simulated runoff to the time of concentration, simulations of the storm of 2 June 77 (Storm No. 6),

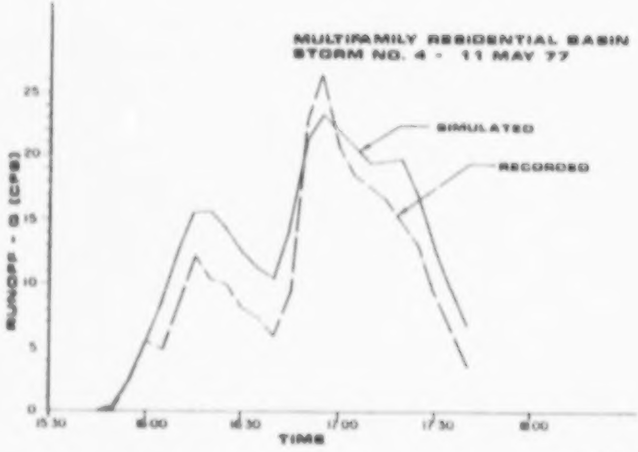


FIGURE 4

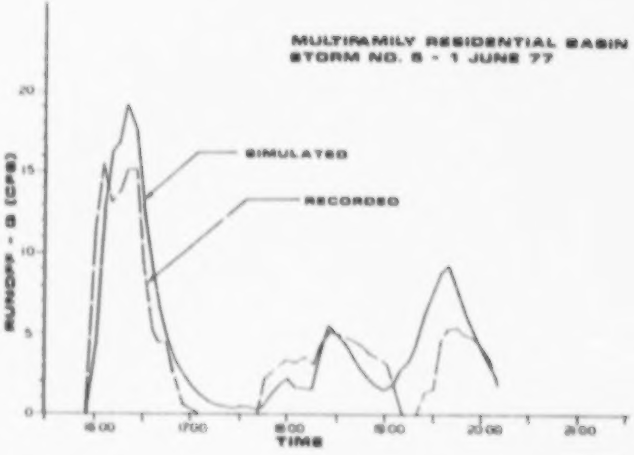


FIGURE 5

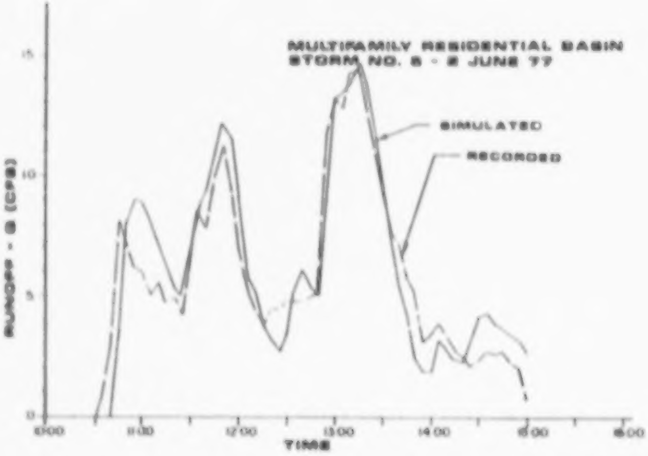


FIGURE 6

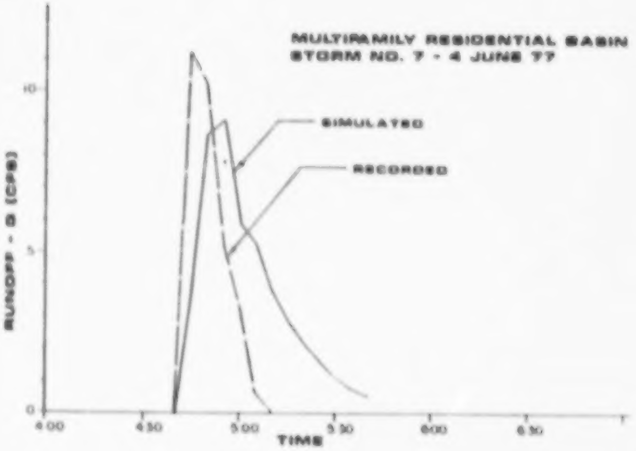


FIGURE 7

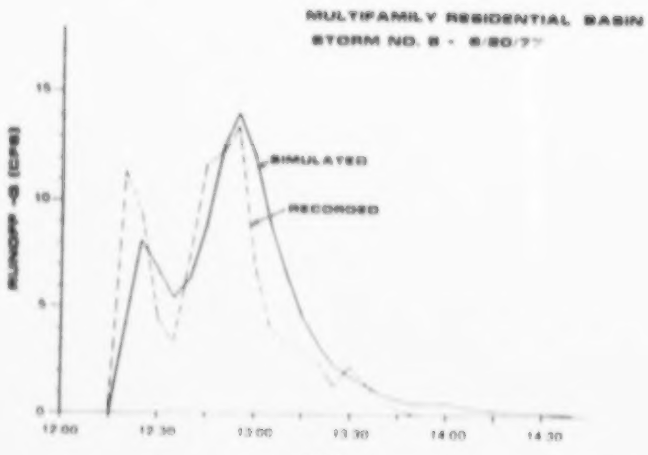


FIGURE 8

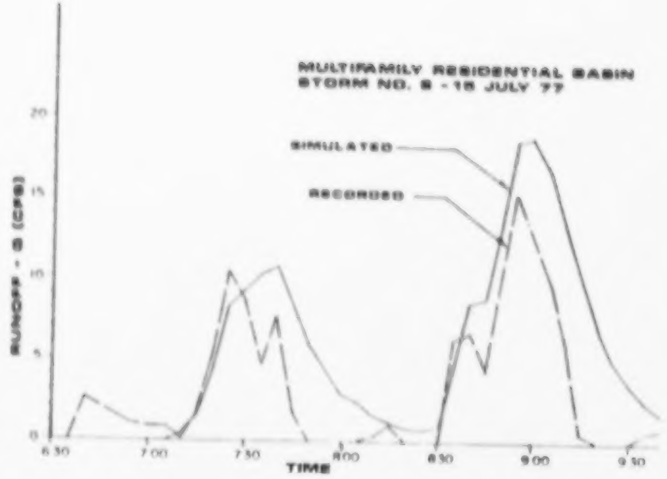


FIGURE 9

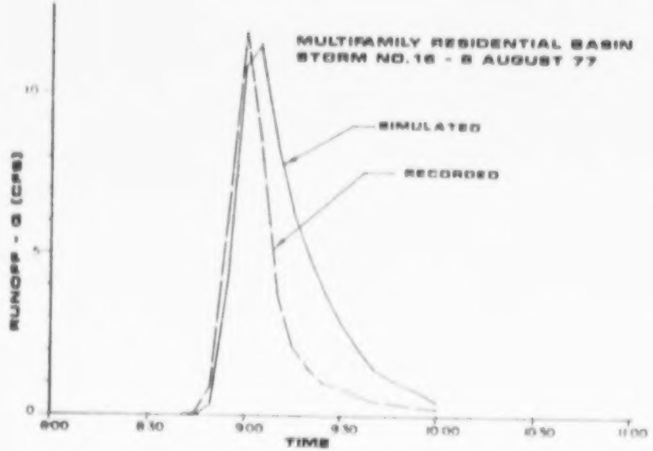


FIGURE 10

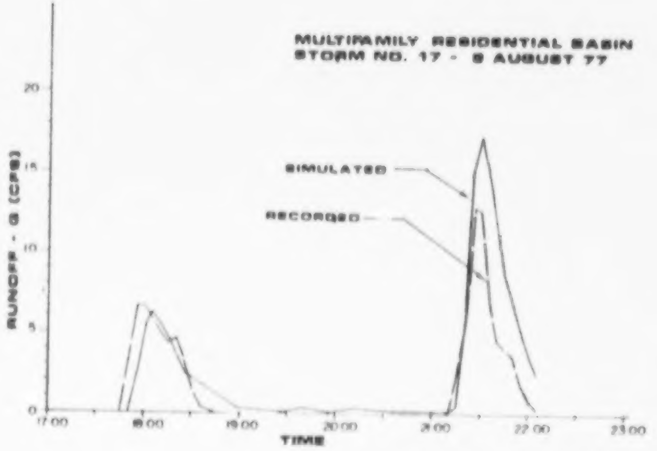


FIGURE 11

were also made for times of concentration of 12 and 18 minutes. As can be seen from Table 6, an approximately 5 percent difference in peaks resulted which differences were not considered significant.

TABLE 5

MODEL VALIDATION RESULTS

MULTIFAMILY RESIDENTIAL BASIN

Storm No.	Storm Date	Rainfall (in.)	Antecedent Moisture Condition	Recorded Runoff (in.)	Simulated Runoff (in.)	Percent Error - Volume	Recorded Peak - Q (cfs)	Simulated Peak - Q (cfs)	Percent Error - Q
4	11 May 77	2.08	4	1.43	1.77	+24	12.23	15.66	+28
5	1 June 77	1.76	1	1.10	1.30	+17	15.13	19.23	+26
6	2 June 77	2.25	4	1.82	1.92	+ 6	14.66	14.67	+ 2
7	4 June 77	0.39	4	0.16	0.26	+63	11.22	9.12	-19
8	20 June 77	1.12	2	0.71	0.74	+ 4	13.47	14.10	+ 5
9	15 July 77	1.12	1	0.67	1.15	+72	10.40	10.62	+ 2
16	8 Aug 77	1.56	3	0.22	0.34	+55	11.91	11.51	- 3
17	8 Aug 77	0.53	3	0.54	0.75	+39	6.77	6.02	-11

TABLE 6

SENSITIVITY ANALYSIS - TIME OF CONCENTRATION

MULTIFAMILY RESIDENTIAL BASIN

STORM NO. 6 - 2 JUNE 77

	T_c (Min.)	First Peak (cfs)	Second Peak (cfs)	Third Peak (cfs)	Volume (in.)
Simulated	12	10.23	13.06	15.33	1.93
Simulated*	15	9.80	12.35	14.67	1.92
Simulated	18	9.23	11.76	14.02	1.91
Recorded	-	8.23	11.36	14.66	1.82

*Actual Condition

Basin No. 2: In all runoff from 13 rainfall events on the USGS's Transportation (Highway) Basin were simulated; the results of which simulations are shown on Figures 12 thru 24. For all events simulated, a time of concentration of 20 minutes (=15 minutes computed pipe flow time + 5 minutes of overland flow time) as previously stated was used. Again, the rainfall events simulated were also essentially random except that again care was taken to insure that these events included a wide range of antecedent moisture condition.

In order to determine the area of the basin actually contributing to flow, runoff from the storm of 28 May 76 (Storm No. 47), P=1.68"-Antecedent Moisture Condition 3 (Rather Dry) was simulated with 100%, 75%, 66.7% and 50% of the pervious area assumed to be contributing. Since rainfall on the nondirectly connected impervious area was assumed in the model to be supplementary rainfall uniformly applied to and distributed over the pervious area, a reduction in the pervious area of 25% (75% of pervious area assumed contributing to runoff) was accompanied by a 25% reduction in the nondirectly connected impervious area. This sensitivity analysis indicated that a reduction in the contributary pervious area by 50% resulted in the best simulation of the actual runoff hydrographs and verified the conclusions made as a result of the visual (field) inspection - that only 50% of the pervious area would contribute runoff from the rainfall events simulated, which assumption was made for the remainder of the simulations made.

As can be observed from Figures 12 thru 24, the simulations, except for the storm of 11 June 75 (Storm No. 53) as shown on Figure 17, were relatively good. The results of these simulations are listed on Table No. 7.

In order to determine the sensitivity of the model (simulated runoff) to the Antecedent Moisture Condition, simulation of the storm of 27 June 1977 (Storm No. 100) as shown on Figure 22 were also made for Antecedent Moisture Conditions 1, 2 and 4 (Bone Dry, Rather Dry and Saturated) in addition to the actual condition (Condition No. 3 - Rather Wet). The results of this sensitivity analysis as shown on Table 8 show differences as much as 100% in the computed peak values and 40% in the computed volumes. Basins with less pervious soils would obviously not be as sensitive to differences in the Antecedent Moisture Condition.

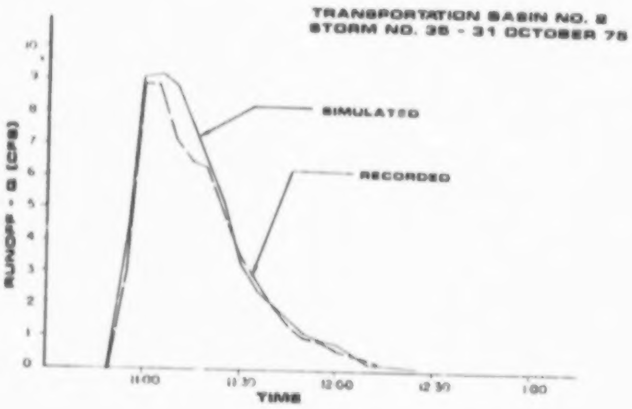


FIGURE 12

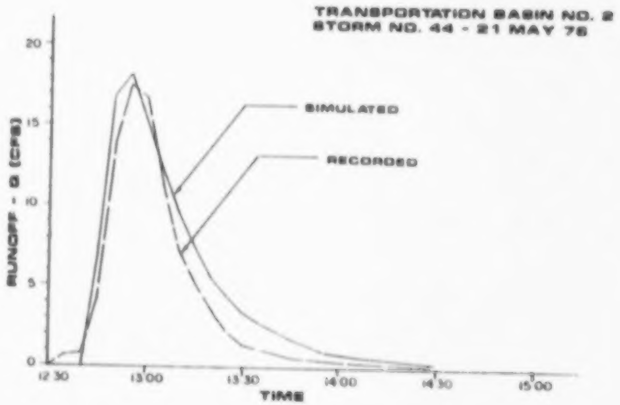


FIGURE 13

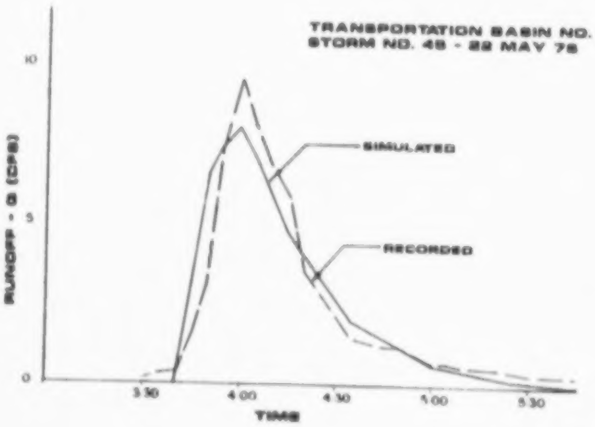


FIGURE 14

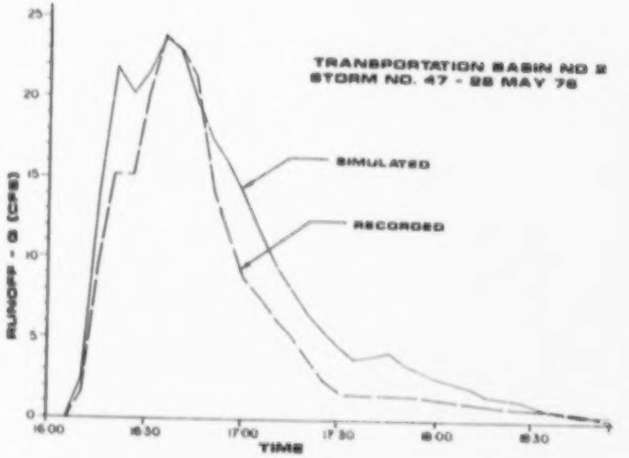


FIGURE 15

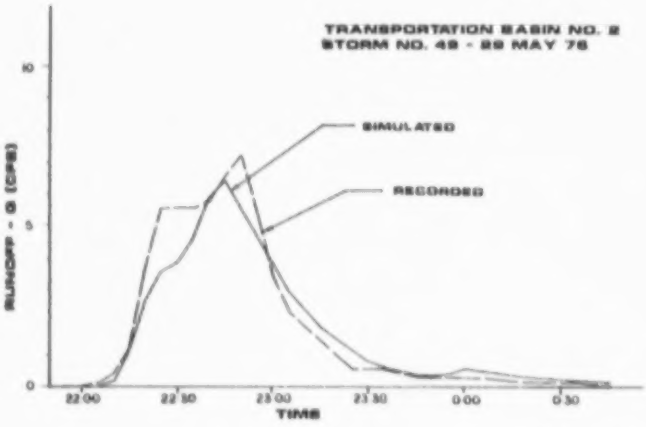


FIGURE 16

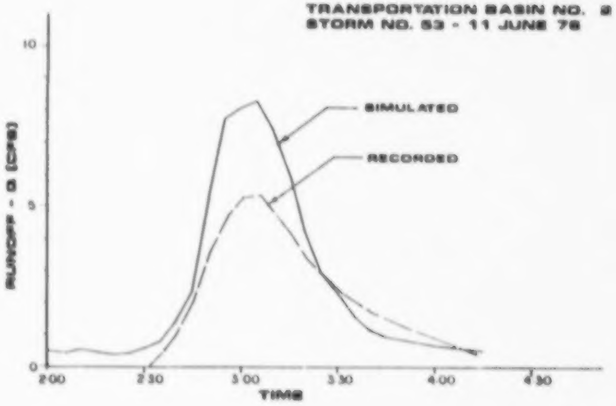


FIGURE 17

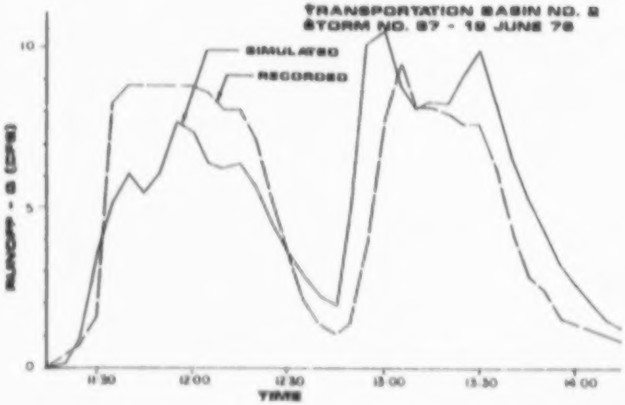


FIGURE 18

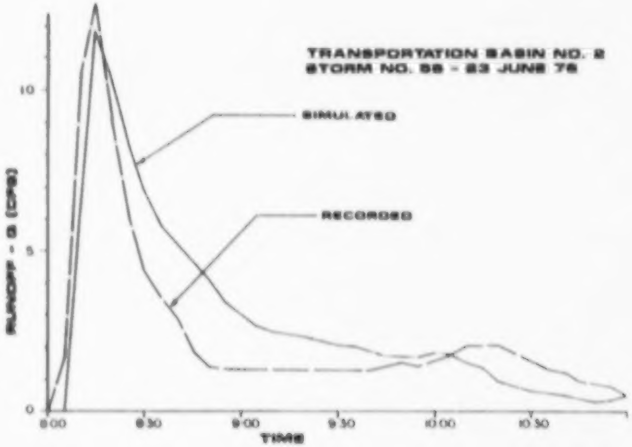


FIGURE 19

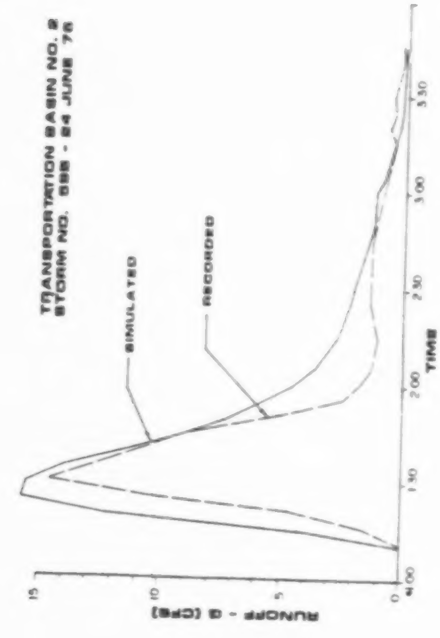


FIGURE 20

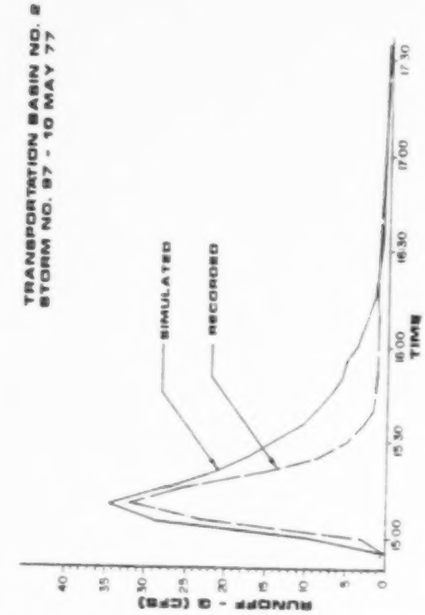


FIGURE 21

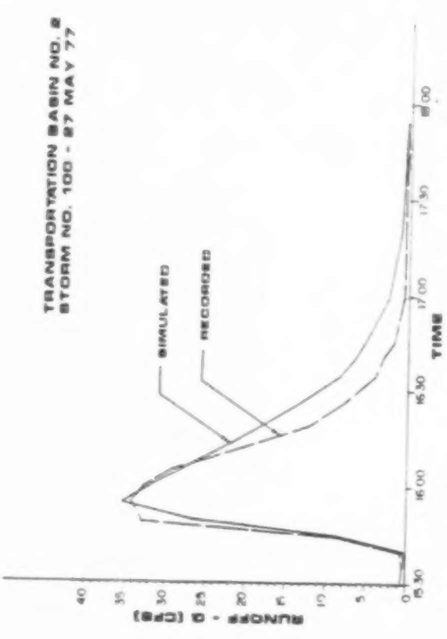


FIGURE 22

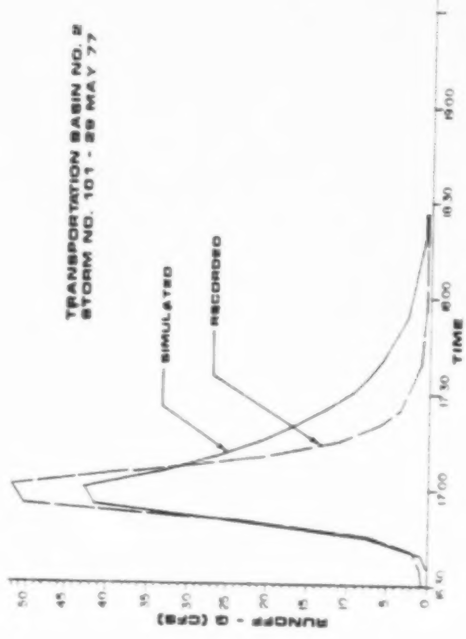


FIGURE 23

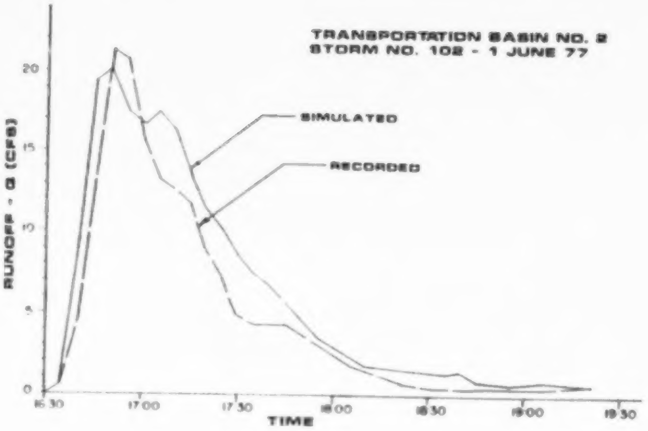


FIGURE 24

TABLE 7

MODEL VALIDATION RESULTS

TRANSPORTATION (HIGHWAY) BASIN

Storm No.	Storm Date	Rainfall (in.)	Antecedent Moisture Condition	Recorded Runoff¹ (in.)	Simulated Runoff¹ (in.)	Percent Error - Volume	Recorded Peak - Q (cfs)	Simulated Peak - Q (cfs)	Percent Error - Q
35	31 Oct 75	0.54	3	0.16	0.17	+06	8.86	9.19	+ 4
44	21 May 76	0.57	4	0.23	0.27	+17	17.30	18.16	+ 5
46	22 May 76	0.47	4	0.16	0.15	- 6	9.50	7.98	-16
47	28 May 76	1.68	3	0.50	0.66	+32	23.69	23.60	0
49	29 May 76	0.57	4	0.15	0.14	- 7	7.32	6.46	-11
53	11 June 76	0.51	4	0.16	0.12	-25	8.22	5.46	-34
57	19 June 76	1.40	2	0.44	0.47	+ 7	8.85	7.65	-14
58	23 June 76	1.08	4	0.33	0.38	+15	12.77	11.88	- 7
59B	24 June 76	0.74	4	0.20	0.31	+55	14.57	16.86	+16
97	10 May 77	1.04	3	0.32	0.52	+63	32.05	34.30	+ 7
100	27 May 77	1.37	3	0.53	0.63	+19	34.28	35.29	+ 3
101	29 May 77	0.98	4	0.55	0.66	+20	52.45	42.93	-18
102	1 June 77	1.16	4	0.41	0.51	+24	21.46	20.27	- 6

¹Based on Actual Contributing Area of 36.4 Acres.

TABLE 8

SENSITIVITY ANALYSIS - ANTECEDENT MOISTURE CONDITION

TRANSPORTATION (HIGHWAY) BASIN

STORM NO. 100 - 27 JUNE 77

	Antecedent Moisture Condition Condition	 Description	Peak Flow (cfs)	Volume (in)
Simulated	1	Bone Dry	17.21	0.38
Simulated	2	Rather Dry	22.01	0.44
Simulated*	3*	Rather Wet*	35.29	0.63
Simulated	4	Saturated	45.63	0.84
Recorded	-	--	34.28	0.53

*Actual Condition

Basin No. 3: In all, runoff from five rainfall events on the Malvern Catchment were simulated; the results of which simulation are shown on Figures 25 thru 29.

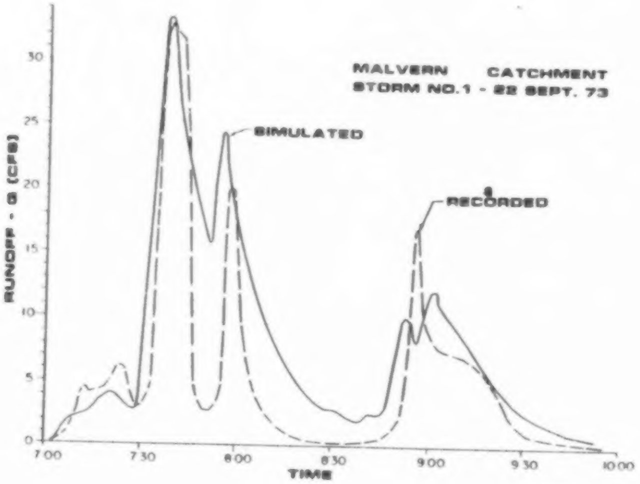


FIGURE 25

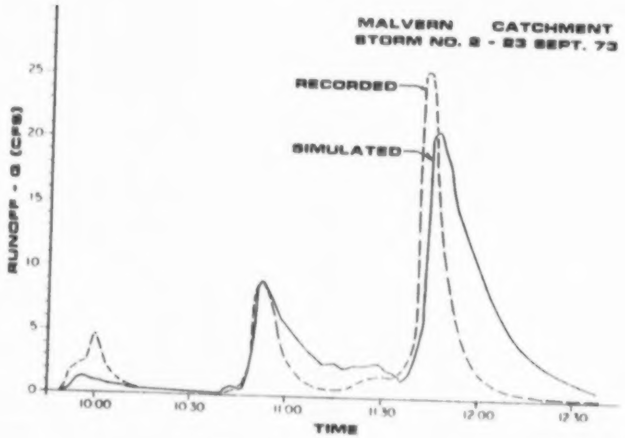


FIGURE 26

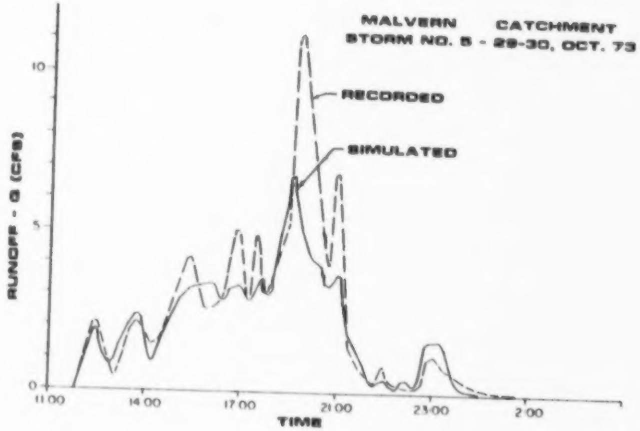


FIGURE 27

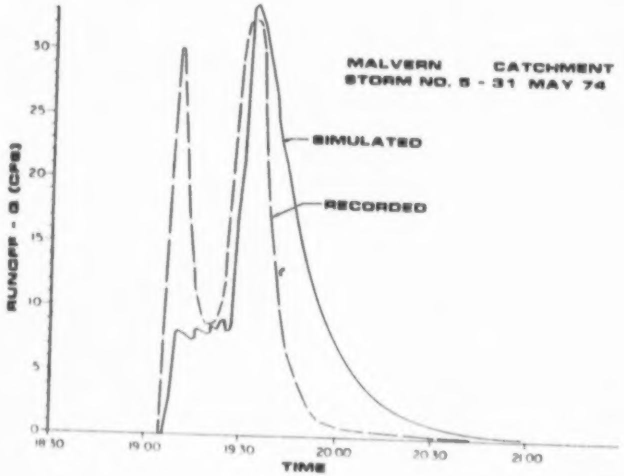


FIGURE 28

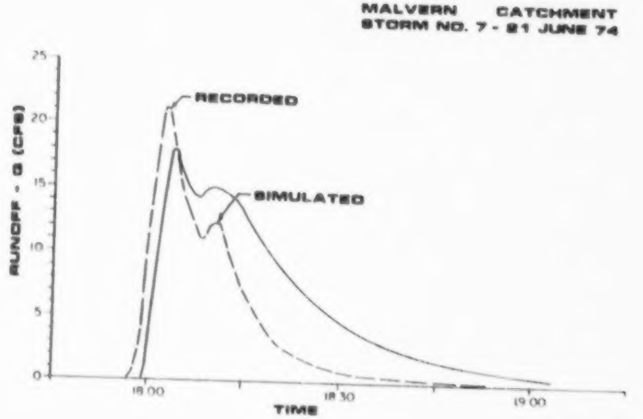


FIGURE 29

As can be observed from these Figures, the simulations were generally fairly good considering that the basin was considered as a whole and not broken down into subbasins.

For all events simulated, it was assumed because of the full development of and steepness of the basin that the total 38.0 acres of pervious area contributed to flow and that in conformance with the assumption in the HNV-SBUH Method, that runoff from the nondirectly connected impervious area (1.63 acres of sidewalks) was supplementary rainfall uniformly applied to and distributed over the pervious area.

For all events simulated the Marsalek infiltration data was used. In applying this data, it was assumed that no water had accumulated in the soil from previous rainfalls (F=0); a Bone Dry (Condition 1) situation existed. Actually, however, the low initial infiltration rate and large decay factor inherent in the Marsalek infiltration data was, for all practical purposes, the equivalent of a constant infiltration rate of 0.52 inch/hour. In all cases, the assumed Marsalek initial abstraction of 0.02 inch rather than the standard 0.1 inch of the program was used.

In all cases, a time of concentration of 15 minutes (9 minutes of pipe flow time at n=0.015 plus 6 minutes of overland flow time) was used.

A sensitivity analysis of the time of concentration for three of the five events simulated showed that variations of three to four minutes did not seriously affect the computed peak flows and had little or no effect on hydrograph shape or volume.

Basin No. 4: In all, runoff from six rainfall events on the Boneyard Creek Basin were simulated; the results of which simulations are shown on Figures 30 thru 35. For all events simulated, a time of concentration of 60 minutes and the reduced (actually contributing) areas as determined by Terstriep and Stall were used.

As can be observed from Figures 30 thru 35, the simulations, except for the storm of 7/2/65 (Storm No. A-24) as shown on Figure 35, were generally quite good considering that the basin was taken as a whole (not subdivided). Also, the simulated hydrographs do not include a base flow which varied from about 2 cfs at the start of the various storms to about 10 cfs at their conclusion, which would have pushed these hydrographs up and to the right, thus improving the simulations over that shown. This one exception to good simulation was attributed to the fact that the particular storm was of such magnitude (1.9" in 60 minutes) that the capacity of the stormwater conveyance system was obviously overtaxed (storm sewer suppression) with resultant significant roadway ponding. Increasing the time of concentration to 80 minutes gave a fair reproduction of the actual recorded hydrograph. Terstriep and Stall using ILLUDAS (2) computed a peak flow of 722 cfs for this particular rainfall event, also significantly greater than that recorded.

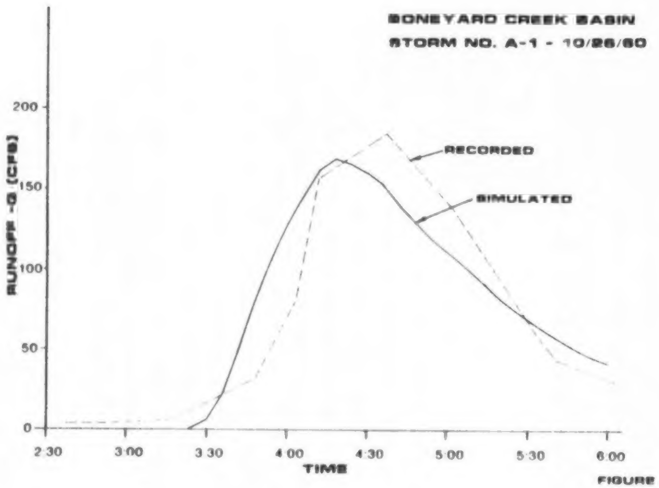


FIGURE 30

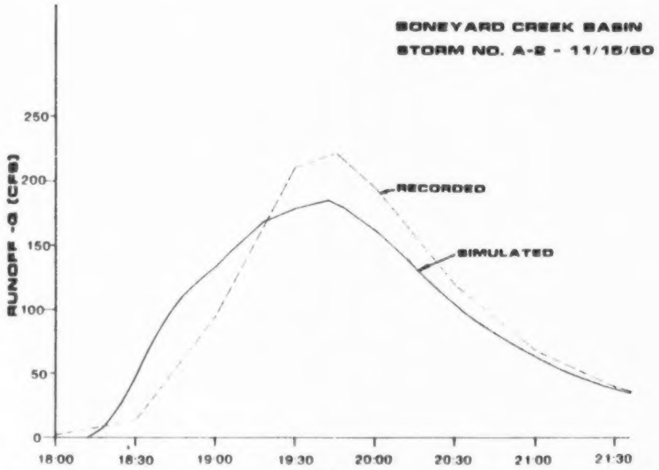


FIGURE 31

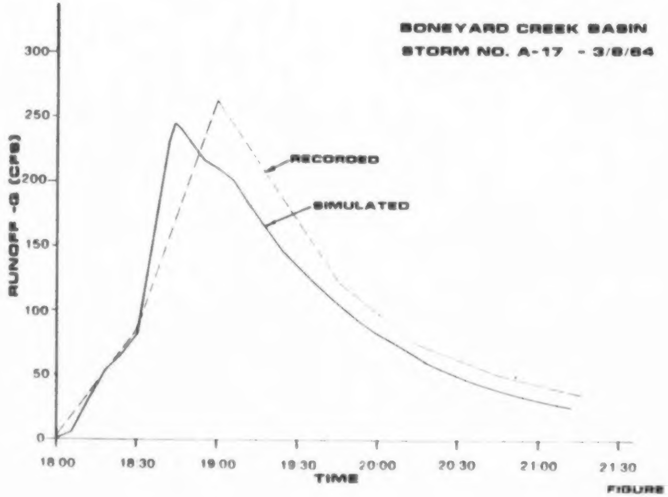


FIGURE 32

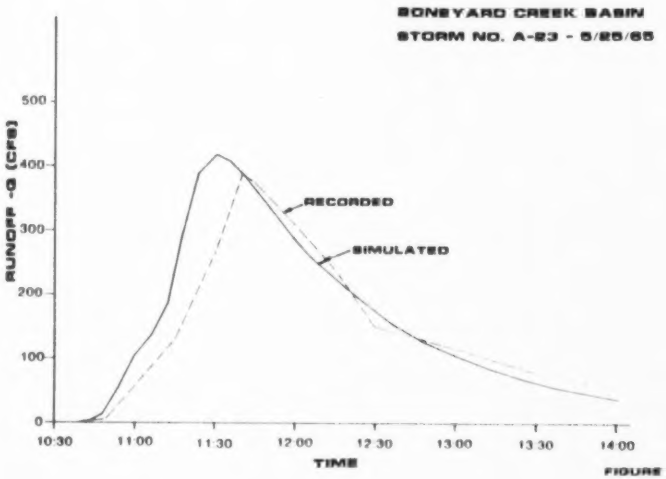


FIGURE 33

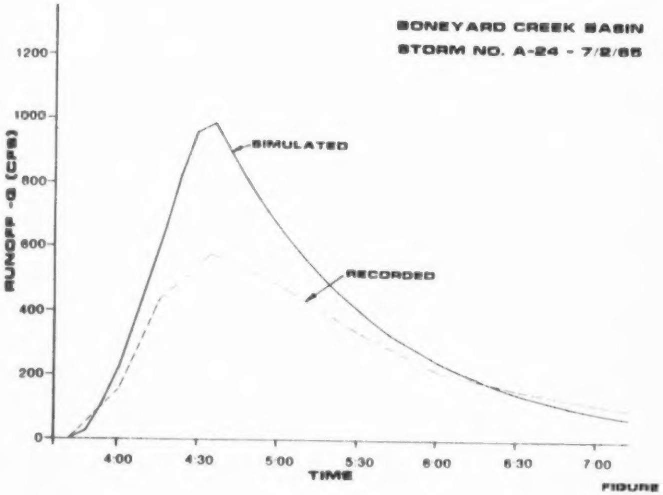


FIGURE 34

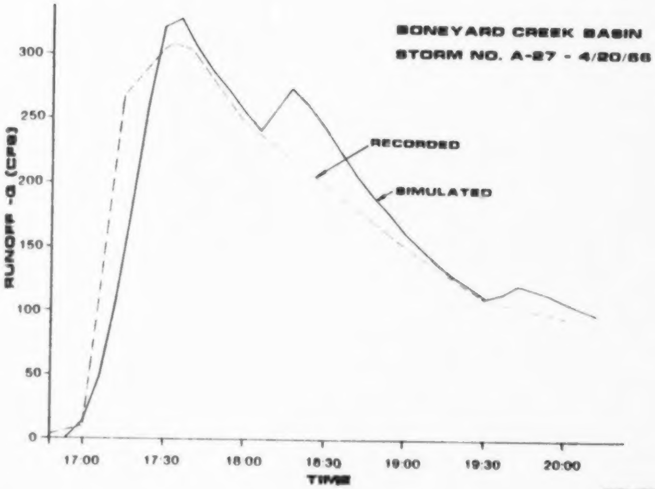


FIGURE 35

Generally acceptable to good simulation of actual runoff events by the HNV-SBUH Method has also been achieved by the writer on the flat 85 acre Hartford Avenue Basin, a single family, storm sewered residential area, and on the 58 acre Brentwood Drive Basin, a fairly steep storm sewered, mixed residential-commercial area, both in Daytona Beach, Florida and both gaged as part of the 208 studies in that area; on the 28 acre Lake Eola Basin in Orlando, Florida, a downtown business-commercial area almost 100% impervious and drained by a storm sewer system; on the 494 acre Cedar Hills Basin near Surry, British Columbia, a residential area essentially drained by open ditches; and on the 97 acre St. Louis Heights Basin, a very steep, storm sewered, residential area in Hawaii. Site specific infiltration data was used for simulating runoff from the Cedar Hills and St. Louis Heights Basins. A simulation of the May 12, 1979 rainfall event over the 28 acre Lake Eola Basin is shown on Figure 36.

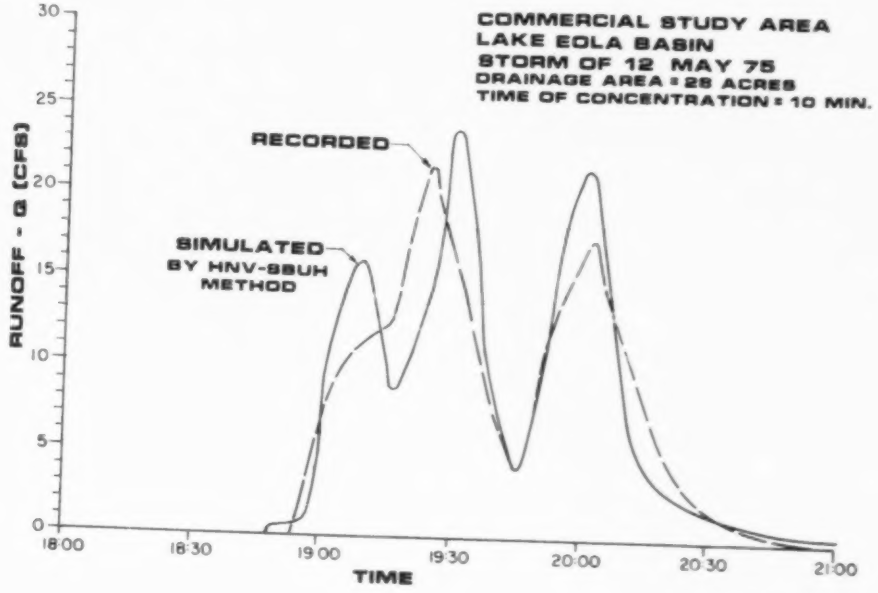


FIGURE 36

Program Descriptions:

Two separate versions for the HNV-SBUH Method are listed at the end of this paper. The first version requires the manual entrance of each rainfall increment. In the second version, all of the rainfall increments are entered at the start of each program.

Hewlett-Packard (HP) Programs: The first HP program, referred to as the HP-67 version although it will operate on the HP-97, requires the use of and can be recorded on a single magnetic card. Initially, after the variables are entered and the program started (Press R/S), the infiltration curve is integrated at intervals of 0.01 hours until the volume of water already stored in the soil (=F) is achieved at which time 0.0000000 appears in the display. As previously stated, each rainfall increment must then be manually entered into the calculator, which, when restarted each time (Press R/S), will compute the ordinate of the runoff hydrograph.

The second HP program, referred to as the HP-97 version, or the HNV-SBUH Method-Automatic Version, although it will also operate on the HP-67, has been separated into two different parts each one of which must be recorded on a separate magnetic card.

In the first part of the program (Card No. 1), the rainfall increments (a maximum of 26) are placed by the program into Storage Registers R11-R19; three each in Storage Registers R11-R18 and two in R19. The maximum rainfall increment that can be placed in any of the registers is 9.99 inches; the minimum is 0.01 inch. The rainfall increments so entered can be recorded on a blank magnetic (data) card for subsequent reuse and thereby avoid the necessity of the individual reentry of same.

In the second part of the program (Card No. 2) the ordinates of the runoff hydrograph are computed. Initially, as previously stated, the area under the infiltration curve is integrated at time intervals of 0.01 hour until the volume of water already stored in the soil when rainfall begins is achieved (=F). At the conclusion of this operation, the calculator will then automatically start to print the time at the end of each rainfall (=ΣΔt) and the ordinate of the runoff hydrograph Q in cfs at that time. The program stops when the ordinate of the runoff hydrograph <0.01 (0.01 displayed).

In this version zero (0.0) is used as a flag in the second part of the program, which precludes the use of zero rainfall increments. In such cases, rainfall increments of 0.01 must be used which small increments will not substantially affect the accuracy of the computed hydrographs.

The volume of runoff in inches (area under the hydrograph) can also be computed by both programs (by pressing fa) which volume is then computed by the equation

$$V = \frac{12 \cdot 3600 \cdot \Sigma Q \cdot \Delta t}{43{,}500 \cdot A} = \frac{0.99274 \cdot \Sigma Q \cdot \Delta t}{A}$$

In both HP versions, the shape (decay) factor K in Horton's equation may be changed from 2 in the programs by deleting 2 from Steps 10 and 65 of the HP-67 program or from Step 188 of Card No. 2 of the HP-97 program and placing another value at these same locations.

Detailed User Instructions precede each of the programs listings.

Conclusions:

1. The HNV-SBUH Method with the four standard infiltration curves (for Soil Groups A, B, C and D) is an effective, easy to use method of simulating acceptable runoff hydrographs from urban drainage basins.

2. Site specific infiltration data would probably have considerably improved the simulations and if possible should be determined and utilized.

3. Runoff hydrographs from residential areas or other urban areas with less than 50% impervious cover (25% directly connected impervious area) are extremely sensitive to antecedent moisture conditions and the infiltration capacity of the soil(s) in the particular basin such that the use of a more sophisticated model to compute runoff hydrographs for design purposes would probably not significantly increase the accuracy of such computed runoff hydrographs such that the use of a more sophisticated model may not be warranted.

4. Small errors in the time of concentration of a particular basin do not appear to significantly affect the accuracy of predicted runoff by the model.

5. Because of the simple input requirements of the HNV-SBUH Method and the ability to quickly compute runoff hydrographs with it on a programmable calculator or on a small microcomputer, its use is extremly economical; hydrographs can be computed in 20 minutes on the programmable calculator and in 10 minutes on a microcomputer by an engineering technician. A calculator program for computing a hydrograph by the HNV-SBUH Method on an HP-33E or HP-25 calculator is available from the writer.

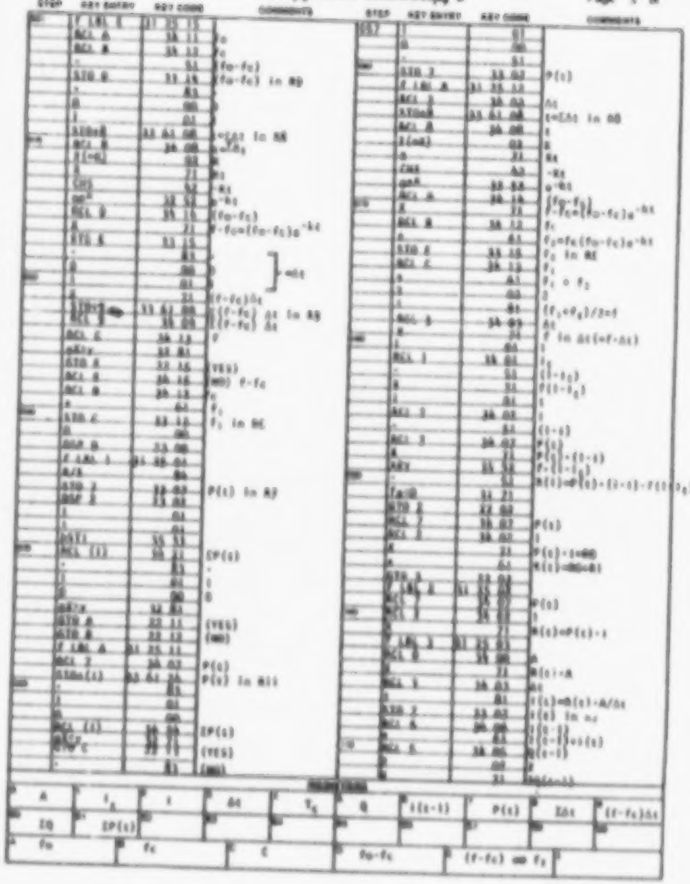
6. The design inflow hydrograph from a major design storm over a basin at the entrance of the hydrograph into the receiving stream computed for the purpose of determining receiving stream size should reflect street ponding (storage) by increasing the time of concentration used to compute such design inflow hydrographs when the conveyance system leading to the receiving stream has been designed for a smaller design storm with resultant pipe sizes too small to adequately convey runoff from the major design storm or when the catch basin capacity is insufficient to permit entrance of all runoff from the major design storm.

User Instructions

[illegible]-SANTA BARBARA URBAN HYDROGRAPH METHOD

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS		OUTPUT DATA/UNITS
1.	Load Program (Sides 1 & 2)				
2.	Initialize (clear all registers)			D	0.00
3.	Input Known Values				
	a) Drainage Basin Area	A (acres)	STO	0	
	b) Total Impervious Portion Drainage Basin	I_t (decimal)	STO	1	
	c) Directly Connected Impervious Portion Drainage Basin	I (decimal)	STO	2	
	d) Time Increment	Δt (hours)	STO	3	
	e) Time of Concentration	T_c (hours)	STO	4	
	f) Initial Infiltration Rate	fo (in/hr)	STO	A	
	g) Final Infiltration Rate	fc (in/hr)	STO	B	
	h) Accumulated Infiltration	F (inches)	STO	C	
4.	Start Program			E	0.00000000
5.	Place First Rainfall Increment in X-Register	$P(t)_1$ (in/Δt)		R/S	Q_1 (cfs)
6.	Continue to place successive Rainfall Increments P(t) in X-Register one at a time until all entered. Then successive Q's	$P(t)_n$ (in/Δt)		R/S	Q_n (cfs)
7.	To compute runoff volume (area under hydrograph)		f	a	Vol (inches)
8.	To compute new hydrograph, go to Step 2				

Program Listing I



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Program Listing II

Page 4 of

STEP	KEY ENTRY	KEY CODE	COMMENTS
		31	[I(t-1)+I(t)-2Q(t-1)]
	RCL 4	34 04	f_s
	2	02	2
	x	71	$2f_s$
	RCL 3	34 03	Δt
	+	61	$2f_s+\Delta t$
	RCL 3	34 03	Δt
	h x⇄y	35 52	$2f_s-\Delta t$
	÷	81	$R=2f_s-\Delta t/2f_s+\Delta t$
	x	71	R[I(t-1)+I(t)-2Q(t-1)]=a
	RCL 5	34 05	Q(t-1)
	+	61	Q(t)=Q(t-1)+a
	STO 5	33 05	Q(t) in R5
	f p⇄s	31 42	
	STO+0	33 61 00	Q(t) in R10
	f p⇄s	31 42	
	RCL 2	34 02	I(t)
	STO 6	33 06	I(t) in R6
	RCL E	34 15	f_2
	STO C	33 13	f_2 becomes f_1
	RCL 5	34 05	Q(t)
	GTO 1	22 01	
	f LBL C	31 25 13	
	0	00	0
	GTO 1	22 01	
	f LBL B	31 25 12	
	CLX	44	Initialization Routine
	f CL REG	31 43	
	f p⇄s	31 42	
	f CL REG	31 43	
	h RTN	35 22	
	g LBL f a	32 25 11	
	RCL 3	34 03	Δt (hours)
	3	03	3
	6	06	6
	0	00	0
	0	00	0
	x	71	Δt (seconds)
	f p⇄s	31 42	
	RCL 0	34 00	ΣQ(t) from R10
	x	71	Volume (ft.³)
	f p⇄s	31 42	
	RCL 0	34 00	A (Acres)
	÷	81	Depth (ft.³/acre)
	4	04	4
	3	03	3
	5	05	5
	6	06	6
	0	00	0
	÷	81	Depth (feet)
	1	01	1
	2	02	2
	x	71	Depth (inches)
	h RTN	35 22	

LABELS | FLAGS | SET STATUS

FLAGS	ON	OFF	TRIG		DISP	
0	☐	■	DEG	■	FIX	■
1	☐	■	GRAD	☐	SCI	☐
2	☐	■	RAD	☐	ENG	☐
3	☐	■			n	2

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User Instructions

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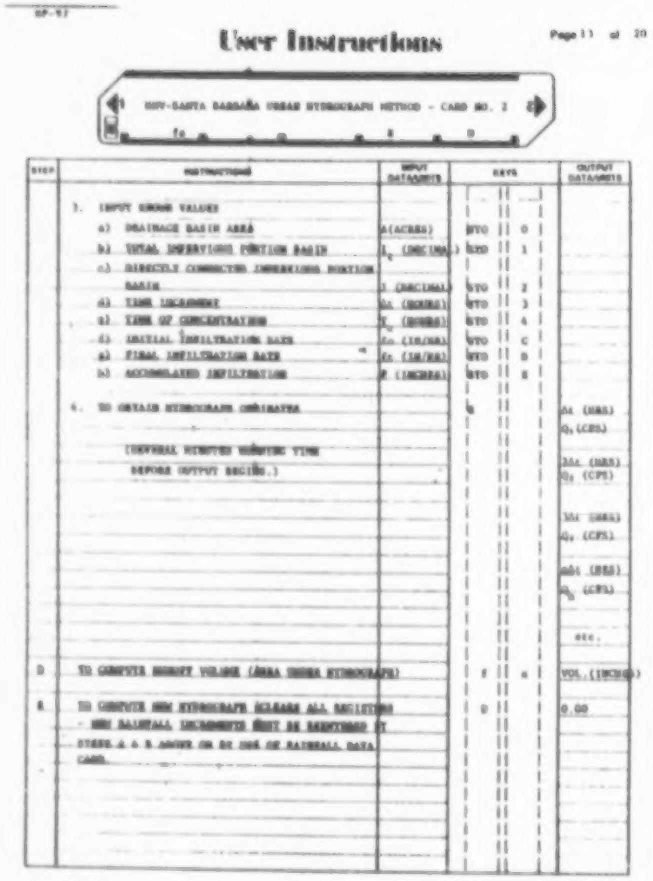
SCS SANTA BARBARA URBAN HYDROGRAPH METHOD - CARD NO. 1

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS		OUTPUT DATA/UNITS
A	ENTER RAINFALL INCREMENTS (MAXIMUM OF 26 IN REGISTERS R11-R19. 3 EACH IN R11-R18 AND 2 IN R19)				
	1. LOAD SIDE 1 OF CARD NO. 1.				
	2. PLACE FIRST RAINFALL INCREMENT IN X-REGISTER.	ΔP₁ (INCHES)	A		COUNT (1.) THEN RE₁
	3. PLACE SECOND RAINFALL INCREMENT IN X-REGISTER	ΔP₂ (INCHES)	R/S		COUNT (2.) THEN RE₁, RE₂
	4. PLACE THIRD RAINFALL INCREMENT IN X-REGISTER	ΔP₃ (INCHES)	R/S		COUNT (3.) THEN RE₁ RE₂, RE₃
	5. PLACE FOURTH RAINFALL INCREMENT IN X-REGISTER.	ΔP₄ (INCHES)	R/S		COUNT (4.) THEN R₄
	6. CONTINUE TO PLACE SUCCESSIVE RAINFALL INCREMENTS IN X-REGISTER	ΔPn (INCHES)	R/S		COUNT, THEN RE's
B	TO RECORD RAINFALL DATA FOR SUBSEQUENT FUTURE USE, PRESS f W/DATA AND PASS BLANK MAGNETIC CARD THRU CALCULATOR. (NOT NECESSARY IF DATA NOT TO BE SAVED.)		f	W/DATA	
C	COMPUTE HYDROGRAPH ORDINATES BY SBUH METHOD.				
	1. LOAD SIDES 1 AND 2 OF CARD NO. 2				
	2. INPUT RAINFALL INCREMENTS INTO CALCULATOR BY LOADING BOTH SIDES OF RAINFALL DATA CARD - ONLY IF RAINFALL INCREMENTS NOT ENTERED IN STEPS A & B ABOVE.	ΔP's (INCHES)			



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User Instructions

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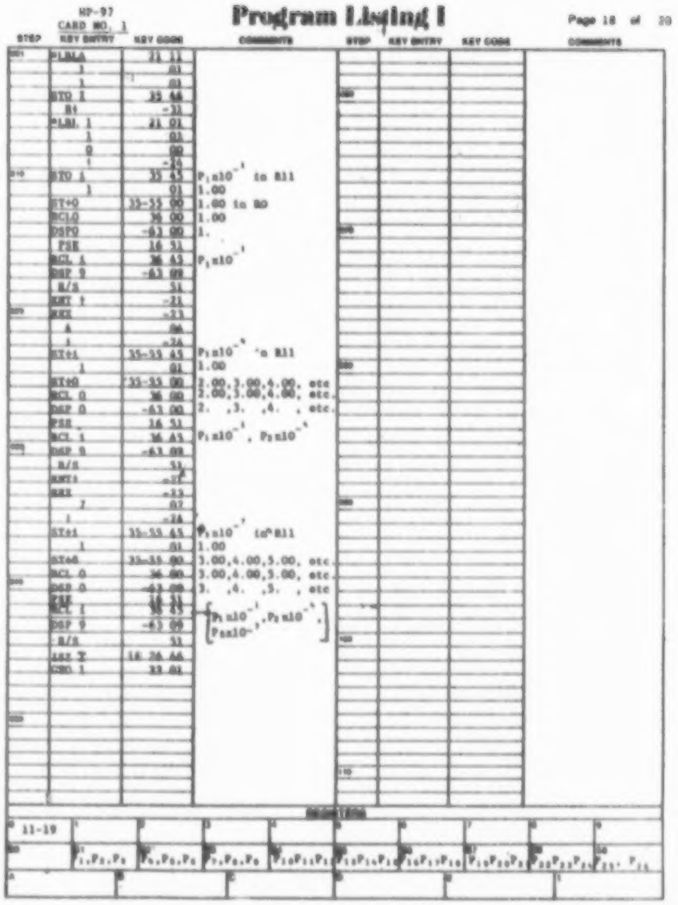
SBUH-SANTA BARBARA URBAN HYDROGRAPH METHOD - CARD NO. 2

fo

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS	OUTPUT DATA/UNITS
3.	INPUT STORM VALUES			
	a) DRAINAGE BASIN AREA	A(ACRES)	STO 0	
	b) TOTAL IMPERVIOUS PORTION BASIN	I_t (DECIMAL)	STO 1	
	c) DIRECTLY CONNECTED IMPERVIOUS PORTION BASIN	I (DECIMAL)	STO 2	
	d) TIME INCREMENT	Δt (HOURS)	STO 3	
	e) TIME OF CONCENTRATION	T_c (HOURS)	STO 4	
	f) INITIAL INFILTRATION RATE	fo (IN/HR)	STO C	
	g) FINAL INFILTRATION RATE	fc (IN/HR)	STO D	
	h) ACCUMULATED INFILTRATION	F (INCHES)	STO E	
4.	TO OBTAIN HYDROGRAPH ORDINATES		B	Δt (HRS) Q_1 (CFS)
	(SEVERAL MINUTES RUNNING TIME BEFORE OUTPUT BEGINS.)			2Δt (HRS) Q_2 (CFS)
				3Δt (HRS) Q_3 (CFS)
				nΔt (HRS) Q_n (CFS)
				etc.
5	TO COMPUTE RUNOFF VOLUME (AREA UNDER HYDROGRAPH)		f a	VOL.(INCHES)
6	TO COMPUTE NEW HYDROGRAPH (CLEARS ALL REGISTERS) - NEW RAINFALL INCREMENTS MUST BE REENTERED BY STEPS 4 & 5 ABOVE OR BY USE OF RAINFALL DATA CARD.		D	0.00



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CARD NO. 1

Program Listing I

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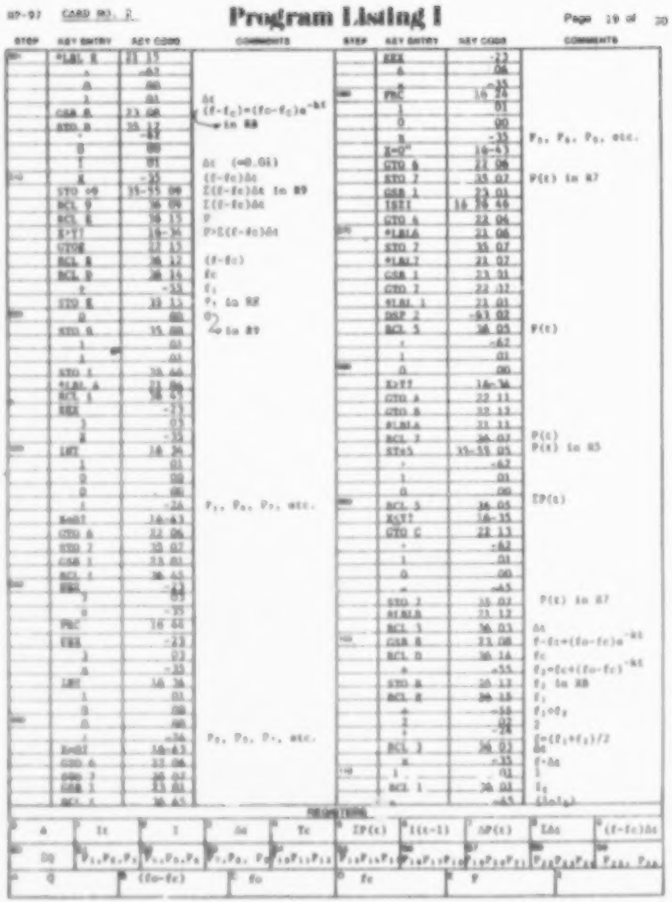
STEP	KEY ENTRY	KEY CODE	COMMENTS
001	*LBLA	21 11	
	1	01	
	1	01	
	STO I	35 46	
	R↓	-31	
	*LBL1	21 01	
	1	01	
	0	00	
	÷	-24	
010	STO i	35 45	P_1x10^{-1} to R11
	1	01	1.00
	ST+0	35-55 00	1.00 to R0
	RCL0	36 00	1.00
	DSP0	-63 00	1.
	PSE	16 51	
	RCL i	36 45	P_1x10^{-1}
	DSP 9	-63 09	
	R/S	51	
	ENT↑	-21	
020	EEX	-23	
	4	04	
	÷	-24	
	ST+i	35-55 45	P_2x10^{-4} to R11
	1	01	1.00
	ST+0	35-55 00	2.00,3.00,4.00, etc
	RCL 0	36 00	2.00,3.00,4.00, etc.
	DSP 0	-63 00	2. ,3. ,4. , etc.
	PSE	16 51	
	RCL i	36 45	P_1x10^{-1}, P_2x10^{-4}
030	DSP 9	-63 09	
	R/S	51	
	ENT↑	-21	
	EEX	-23	
	7	07	
	÷	-24	
	ST+i	35-55 45	P_3x10^{-7} to R11
	1	01	1.00
	ST+0	35-55 00	3.00,4.00,5.00, etc.
	RCL 0	36 00	3.00,4.00,5.00, etc.
040	DSP 0	-63 00	3. ,4. ,5. , etc
	PSE	16 51	
	RCL i	36 45	[P_1x10^{-1}, P_2x10^{-4}, P_3x10^{-7}]
	DSP 9	-63 09	
	R/S	51	
	ISZ I	16 26 46	
	GTO 1	22 01	
050			
060			
070			
080			
090			
100			
110			

REGISTERS

0	1	2	3	4	5	6	7	8	9
11-19									

S0	S1	S2	S3	S4	S5	S6	S7	S8	S9
	P_1,P_2,P_3	P_4,P_5,P_6	P_7,P_8,P_9	$P_{10}P_{11}P_{12}$	$P_{13}P_{14}P_{15}$	$P_{16}P_{17}P_{18}$	$P_{19}P_{20}P_{21}$	$P_{22}P_{23}P_{24}$	P_{25}, P_{26}

A	B	C	D	E	I



HP-97 CARD NO. 2

Program Listing I

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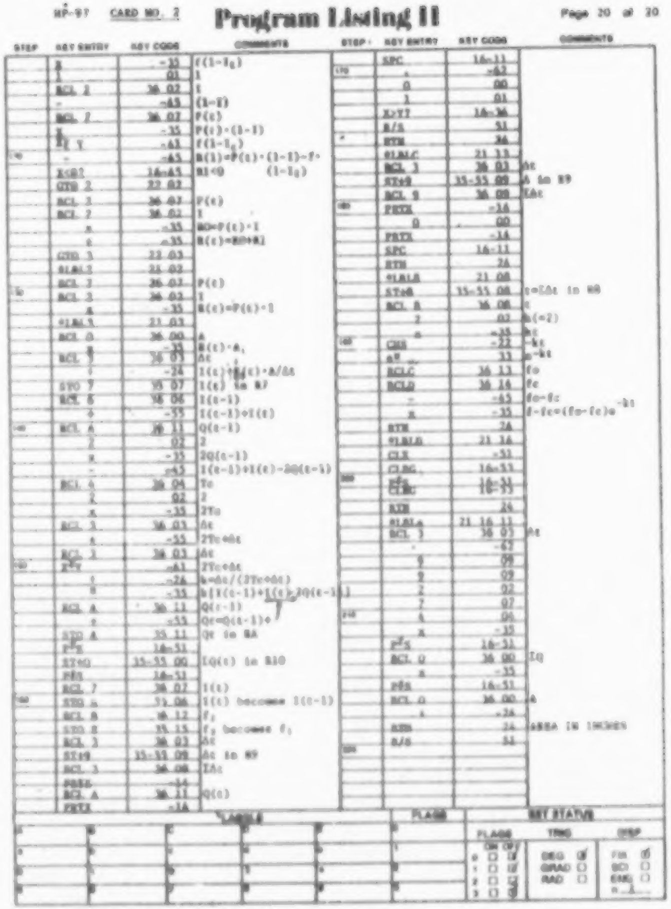

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Program Listing II

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DETENTION LAKE APPLICATION IN MASTER DRAINAGE PLANNING

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TORONTO, CANADA

SUMMARY

This paper illustrates that the Stormwater Lake concept can be applied to obtain an optimum urban drainage system design. The proposed lake system converts the existing sloughs into a system of aesthetic urban lakes interconnected by controlled-release pipes. Computer Models, HYMO, STORM and SWMM were used for simulating the various hydrologic conditions and testing the hydraulic responses within the system. The results of modelling were further utilized to optimize the number, size and layout of the proposed lake system with respect to economics, design, hydraulic operation and development constraints.

1.0 INTRODUCTION

The application of Stormwater Management concepts has gained increasing popularity in the design and analysis of urban drainage systems, and the City of Edmonton has taken a leading role in the application of these new techniques in Canada. A Stormwater Drainage Analysis for the entire north and northeast Edmonton Development Area of which the present study area (Land Section 34 and its vicinity) is a part, was conducted by James F. MacLaren Ltd. (JFM) in

1978 (Ref. 1) and subsequently a paper related to this report was published by A. Fok et al in 1979 (Ref. 2). The lake optimization analysis presented in this paper is founded on the results of these previous studies.

The drainage concept developed in the previous studies is based on the conversion of existing sloughs and depressions into a system of aesthetic urban lakes interconnected for controlled drainage. The plan also incorporates the dual drainage concept of major and minor system flow. Storm sewers of conventional design will drain to the lakes to effect runoff control.

As the present study area is proceeding toward development, a final Stormwater Management Report was required in support of the detailed development plans for City approval. Proctor Redfern Butler & Krebes Limited (PRB&K) was commissioned to conduct this study at the request of the Section 34 owners group. With the knowledge of the detailed land use and neighbourhood layout for the area, this study refined and optimized the number, location, and sizing of Stormwater Management facilities from a land development point-of-view as well as satisfying the City's criteria for flood protection and operation.

This paper focuses on the optimizing of the proposed lake system within the study area. However, the original Stormwater Management concepts, which apply to the whole north and northeast Edmonton development area, are also discussed in some detail to give a complete illustration of this Stormwater Management application.

2.0 ASSESSMENT OF ALTERNATIVE STORMWATER MANAGEMENT CONCEPTS

2.1 Description of the North and Northeast Development Area

The total area of the development located in north and northeast Edmonton (Figure 1) is about 3050 ha. A north-south ridge of land divides the area into an eastern and a western watershed. The 1350 ha eastern watershed drains directly to the North Saskatchewan River. The 1700 ha western watershed (the 'Lake District') is dotted with sloughs and drains southward into the existing Kennedale District Sewer System which services a 3200 ha residential area. Stormwater runoff is now temporarily

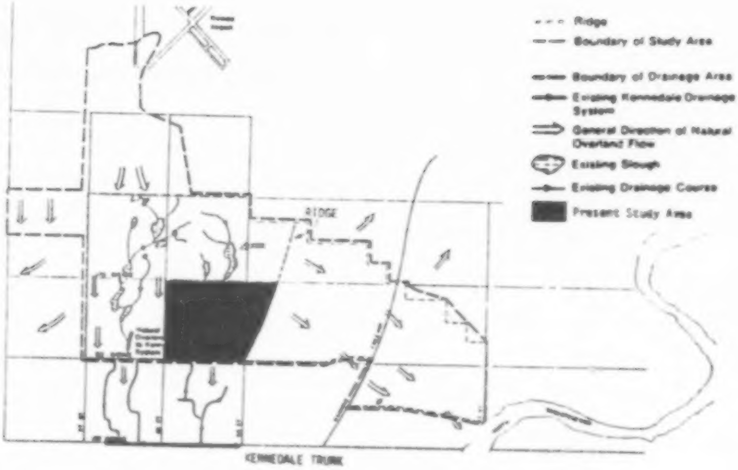
Ridge
Boundary of Study Area
Boundary of Drainage Area
Existing Kennedale Drainage System
General Direction of Natural Overland Flow
Existing Slough
Existing Drainage Course
Present Study Area
RIDGE
KENNEDALE TRUNK

FIG. 1 NATURAL DRAINAGE CONDITIONS

retained in natural sloughs and depressions throughout the watershed. However, spring snowmelt and heavy summer storm runoff can cause very significant flow to the downstream Kennedale System and flooding potential does exist under present conditions.

The existing watersheds (both east and west) are agricultural or vacant land. The proposed development will be predominantly residential with small sectors of light industrial, commercial and institutional lands.

2.2 Description of Alternative Measures

Since the eastern watershed has natural drainage access to the North Saskatchewan River, drainage alternatives for this area included only variations in the location of the main trunk and a comparison between open channel and sewer conveyance with and without in-system detention storage facilities.

However, natural drainage in the western watershed is internally complex due to the sloughs in the central area and externally constrainted by the capacity limits of the southern (Kennedale) receiving system. Three basic Stormwater Management alternatives for controlling runoff from this watershed were distilled from a number of options for more detailed consideration.

- Alternative 1 - Gravity Flow Eastward Through a Tunnel

  This alternative requires the diversion of runoff from its natural southerly direction eastward to the new drainage system east of the ridge. Storage facilities are required locally to limit the size of the tunnel.

- Alternative 2 - Pumping Flows Eastward Over the Ridge

  This alternative is essentially the same as Alternative 1 except that pumping facilities are used to lift the flows over the ridge and eliminate the need for a deep tunnel. It should be noted that this alternative originated from the fact that a pumping station is required anyway to lift sanitary flows over the ridge.

- Alternative 3 - Controlled Release to the Kennedale Storm Sewer System

  This alternative maintains the normal flow direction to the south. The only possible way of maintaining this natural flow direction without severe impact to the downstream area is to strictly control the amount of flow entering the Kennedale System. Detention lakes are therefore required to store the excess runoff from the area. The concept of Alternative 3 is illustrated in Figure 2.

2.3 Comparison of Alternatives

In order to facilitate the comparisons of the alternatives, the merits and demerits of each alternative were summarized in Table 1.

It was found that Alternative 3 results in a considerable cost reduction over the other alternatives due to the elimination of the tunnel and oversizing of the eastern trunk sewers. Furthermore, Alternative 3 permits the expenditures for drainage to be staged in accordance with the rate of development. New storage ponds can be constructed as required to control flows from each new development. Staging is also possible for the investments in the sanitary sewer system. In addition, storage ponds have environmental advantages over a system that would pipe runoff directly into the river. For example, the ponds will improve the urban aesthetics, enhance groundwater recharge, improve water quality

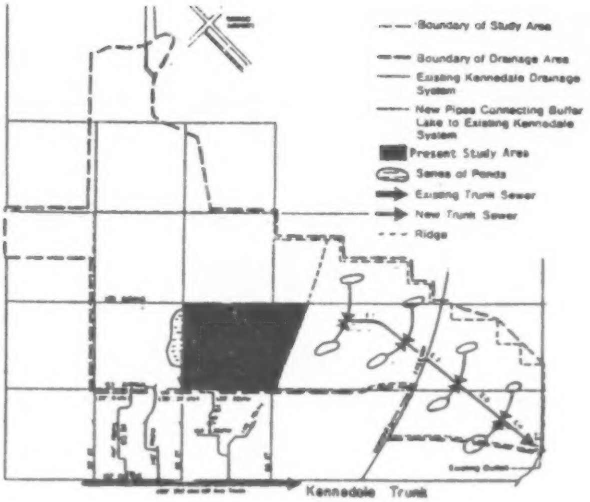
Boundary of Study Area
Boundary of Drainage Area
Existing Kennedale Drainage System
New Pipes Connecting Buffer Lake to Existing Kennedale System
Present Study Area
Series of Ponds
Existing Trunk Sewer
New Trunk Sewer
Ridge
Existing Outlet
Kennedale Trunk

FIG. 2 SCHEMATIC OF ALTERNATIVE 3 - CONTROLLED RELEASE

TABLE 1

COMPARISON OF ALTERNATIVES

Alternative	Advantage	Disadvantage
1) Conventional Relief with Detention	1) Simplicity in conception and operation. 2) Sanitary and storm sewers in same alignment.	1) Reverse natural flow direction. 2) No overland flow outlet for major system. 3) Most expensive. 4) Heavy front end expenditure for the tunnel. 5) Staging impossible.
2) Pumping Relief with Detention	1) Pumping capacity can be staged. 2) Sanitary and storm sewers in same alignment. 3) Slightly less expensive than Alternative 1.	1) Reverse natural flow direction. 2) No overland flow outlet for major system. 3) Additional operational requirement.
3) Controlled Release with Detention	1) Follow natural drainage direction. 2) Least expensive. 3) Can be staged. 4) Outlet of overland flow available.	1) Additional protection measure to downstream development required.

and provide reservoirs for possible use in irrigating parklands adjacent to the ponds during dry periods.

Therefore, Alternative 3 was selected as the most acceptable drainage scheme.

2.4 Storage of the Proposed Lake System (Total System Requirement)

One of the design criteria for the proposed lake system was to provide high level of flood protection and runoff control within the study area and in the existing downstream development in the City. As such, the lake system must provide adequate storage to accommodate runoff volumes from critical hydrological events. The system was tested for several conditions as discussed below.

- The 100 Year - 6 Hour Duration Storm is the design level of flood protection required by the City. As seen from Figure 3A, uncontrolled drainage after development would result in a significant increase in the peak rate and volume of runoff. However, the proposed lake storage volume of 10.3 cms-day will reduce outflow south to the Kennedale Sewer System to a nominal rate of 1.42 cms and no overland flow will occur. By comparison, the present conditions will produce large overland flow rates in the order of 42.5 cms for this event.

- The July 14-15, 1937 Storm is the largest storm on record, when 163 mm rainfall fell in 48 hours. The proposed lake system (10.3 cms-day) was analysed for this event. It was found that there would not have been a significant reduction in the peak discharges for this event but the duration of the overland flow would have been substantially reduced (Figure 36).

  Furthermore, the return frequency of this event is estimated to be about 1/250 years. As economic considerations preclude designing for such an extremely rare occurrence, and since the potential flooding for the proposed conditions would improve somewhat over the existing conditions, this storm was not considered for the determination of the storage requirement.

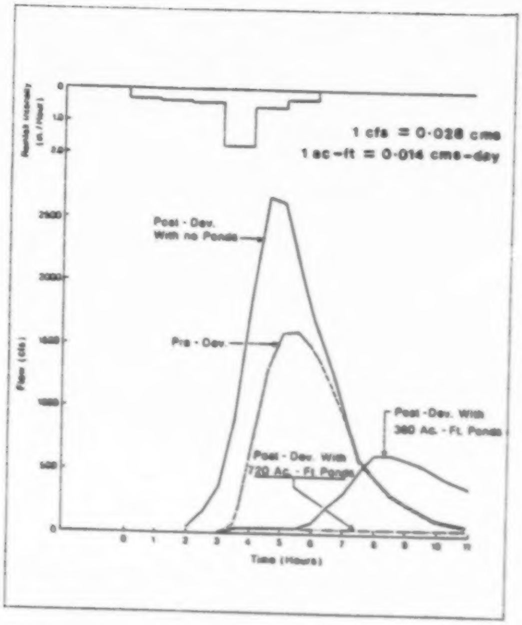

FIG. 3a OVERFLOW HYDROGRAPHS FOR 100-YEAR 6-HOUR DESIGN STORM

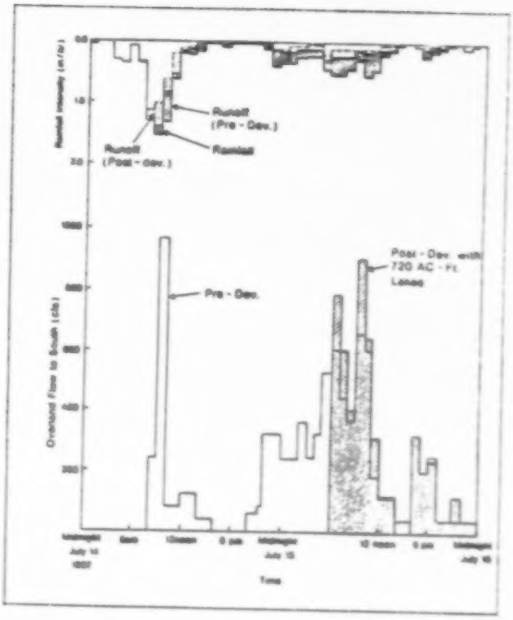

FIG. 3b OVERLAND FLOW HYDROGRAPH OF JULY 14,1937 STORM

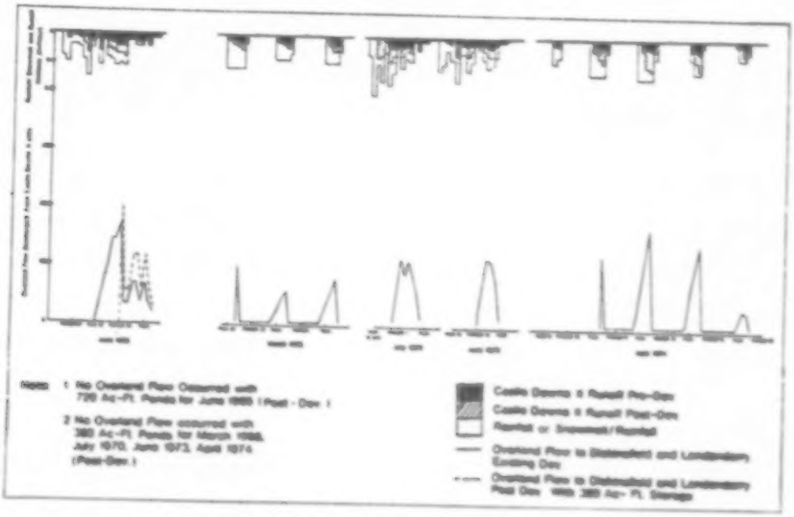

FIG. 3c OCCURENCE OF OVERLAND FLOW CONDITION, 1961-1977

- The 17 Years of Rainfall Data from 1961 - 1977 contained one of the largest rainfall months in record (June, 1965). These data were used to test the system for consecutive small events and snowmelt/rainfall events. It was found that these events are not critical as no overflows were predicted for this period from the proposed lake system (10.3 cms-day design storage). This represents a definite improvement compared to existing conditions for which five significant overland flow events were predicted for this data (Figure 36).

From these analyses it can be concluded that the 10.3 cms-day lake storage system is adequate for the whole 1700 ha Lake District. The next stage of the study was directed to a better definition of the system with respect to its components, operation for different hydrologic conditions and integration with the downstream system.

3.0 OPTIMIZATION OF THE LAKE SYSTEM

3.1 Development Plan of the Study Area

The present study area (Land Section 34 and its vicinity) consists of a 307 ha drainage area. It is located at the southwest corner of the Lake District (Figure 1). The general flow pattern and the relevant land use data which may affect the drainage design are summarized in Figures 4 and 5.

The original 1978 study proposed a total of seven lakes for this area. Because of the newly adopted maintenance and operation criteria, the refinement of development plans and cost-benefit considerations, the number, size and location of the lakes in this area were to be optimized to fulfill the design criteria as discussed below.

3.2 Design Criteria

In order to optimize the use of the lake system for flood protection and community benefits, several design criteria were adopted:

1. The lakes should be easily accessible to the community.

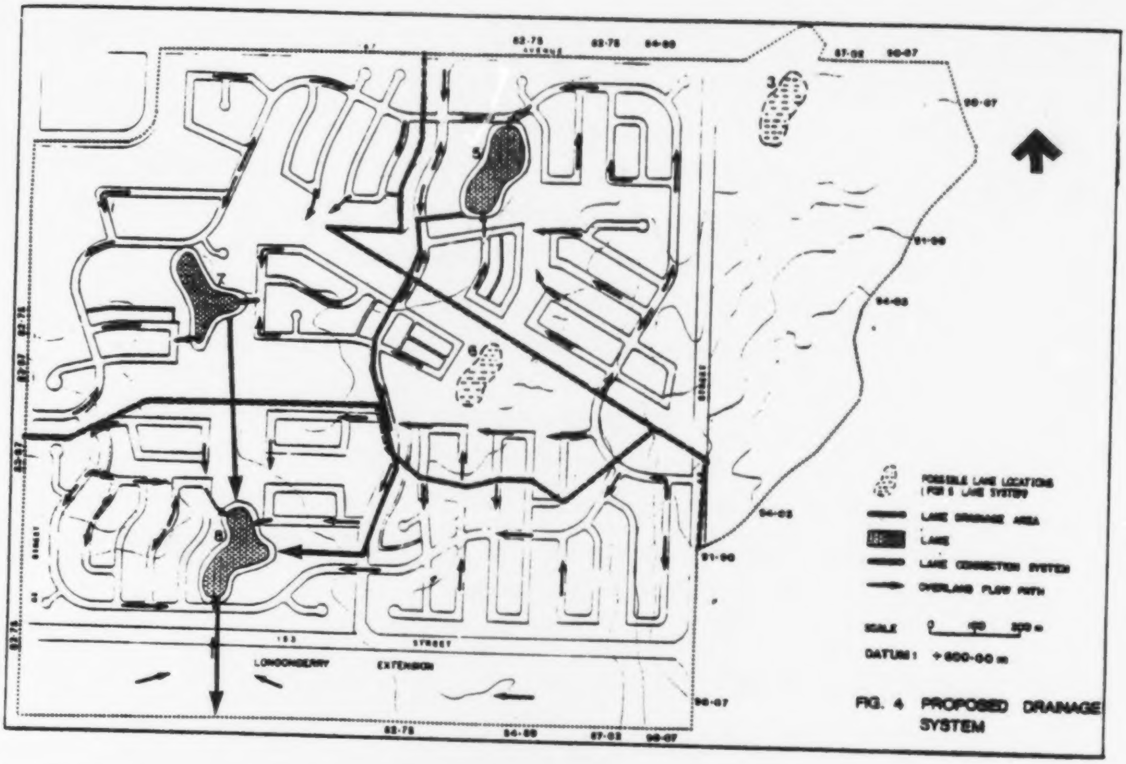
AVENUE
3
5
6
7
8
STREET
LONDONDERRY EXTENSION
POSSIBLE LAKE LOCATIONS (FOR 8 LAKE SYSTEM)
LAKE DRAINAGE AREA
LAKE
LAKE CONNECTION SYSTEM
OVERLAND FLOW PATH
SCALE 0 100 200 m
DATUM: +600-00 m

FIG. 4 PROPOSED DRAINAGE SYSTEM

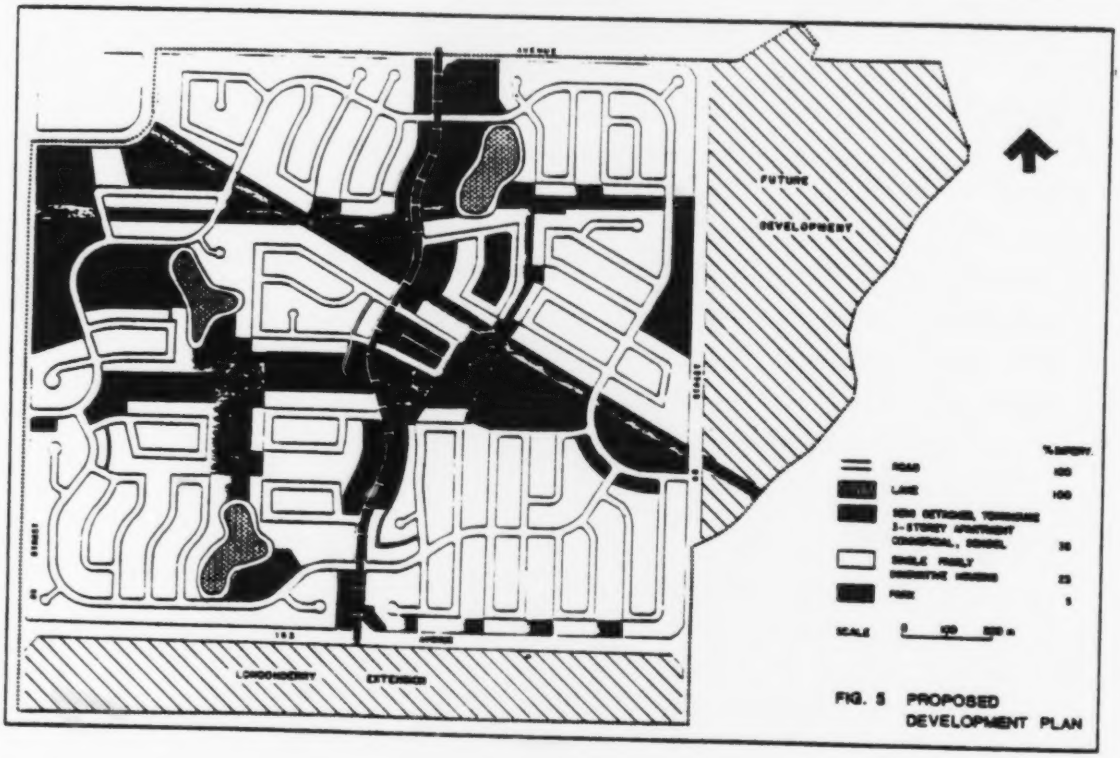
AVENUE
FUTURE DEVELOPMENT
STREET
LONDONDERRY EXTENSION
% IMPERV.
ROAD 100
LAKE 100
SEMI DETACHED, TOWNHOUSE 3-STOREY APARTMENT COMMERCIAL, SCHOOL 35
SINGLE FAMILY INNOVATIVE HOUSING 25
PARK 5
SCALE 0 100 200 m

FIG. 5 PROPOSED DEVELOPMENT PLAN

2. The lakes are to be located on the existing sloughs and depressions.
3. The size of the lakes should be proportional to the local drainage area.
4. A balance design is required for the lake system. The outflow from each lake should be proportional to the drainage area and remain constant during the storm. This eliminates the need for oversizing the downstream lakes and facilitates staging and cost sharing.
5. For storms larger than 1/100 year storm, the sharing flooded area should be confined to the vicinity of the lakes.
6. The number of lakes should be minimized to provide easier maintenance.
7. The area of the lake should not be less than 1.6 ha at permanent water level. (P.W.L.)

3.3 Results of Analysis for Various Lake Systems

- Five Lake System (Equal Lake Size)

One of the alternative lake systems for the study area is five lakes (Figure 4). Each lake would be 1.6 ha in size at P.W.L. for a total area of 8 ha. Results of analysis indicated that the equal sizes of lakes for tributary areas of different sizes would result in unequal maximum water depths and variable flow rates in the interconnected pipes during storms. Thus a balanced design of the lake system for this alternative cannot be achieved and, this alternative was not considered further.

- Five Lake System (Balanced Design)

In order to achieve a balanced design, (i.e. a uniform average lake depth) the size of each lake was made approximately proportional to its contributing drainage area (Figure 4) with a target maximum water depth of 2 m. Preliminary analysis indicated that a total lake surface area of 5.2 ha would be adequate. The resulting hydraulic response of the system is summarized below:

Lake No.	Size (ha)	Runoff Volume (cms-day)	Maximum Depth of Water Above P.W.L.(m)
3	0.9	0.30	2.04
5	1.0	0.33	2.04
6	0.33	0.11	1.95
7	1.15	0.38	2.13
8	1.74	0.55	2.10
Total	5.20	1.67	

These results show that the 5.2 ha, Five Lake Alternative can provide adequate 1/100 year flood protection in a balanced system. However, 4 of the 5 lakes are smaller than the 1.6 ha requirement and Lake 6 is only 0.33 ha. Hence, the Five Lake System was not considered further.

- Three Lake System (Balanced Design)

  The location of each lake and its drainage sub-watershed for the Three Lake System are shown in Figure 4. Lake 5 will have a 2.0 ha surface area while Lakes 7 and 8 have 1.6 ha each. The surface area of Three Lakes are roughly proportional to their tributary area and, as shown in Table 2, maximum water depths are quite close in all lakes for each of the design storms.

TABLE 2

IMPERVIOUSNESS VALUES FOR VARIOUS LAND USES

Types of Land Uses	Roof Area m^2/unit	Density (units/ ha.)	% of Imperviousness
R1 - Single Family Residential	116	3.24	23
R2 - Semi-detach Residential	116	4.86	34
R2A- Townhouse	93	6.07	34
R3 - 3 Storey Apartment	186	10.12	36
Innovative Housing	93	4.05	23
Commercial			35
School			33
Park			5
Roads,Lakes			100

The flow hydrograph and water level variation for the lakes are plotted for the 100 year - 6 hour and July 14-15, 1937 Storms in Figures 6 and 7 respectively. Since the water levels in all lakes rise at the same rate during the design storms, a balanced hydraulic response in the Three Lake System has been achieved.

The basin hydraulic profile in the Three Lake System is plotted in Figure 8. This figure indicates clearly that the system can handle the 100 year storm adequately. With the available freeboard, no flooding is expected to occur even for a storm such as the July 14-15, 1937 storm.

During all of the design storm events under investigation it was found that the lake connection system was conveying a flow rate approximately equal to the corresponding pipe capacity and equal to the required control rates. This further illustrates that a smooth hydraulic response in the proposed lake system can be achieved.

The Three Lake System therefore has the following benefits and hence was recommended for the final design:

A) Required flow control and flood protection is achieved with a smaller lake area.

B) Each lake meets the minimum size requirement of the City.

C) A balanced design is achieved in which lake area and outflow rates are based on the respective tributary areas.

4.0 METHODS OF ANALYSIS

4.1 Hydrology

The HYMO Model which is based in the SCS Method, was used in the 1978 Study for simulating single events for design storms.

The Computer Program STORM was also used to simulate a continuous rainfall record (1960-1967) in which closely spaced consecutive storms occurred.

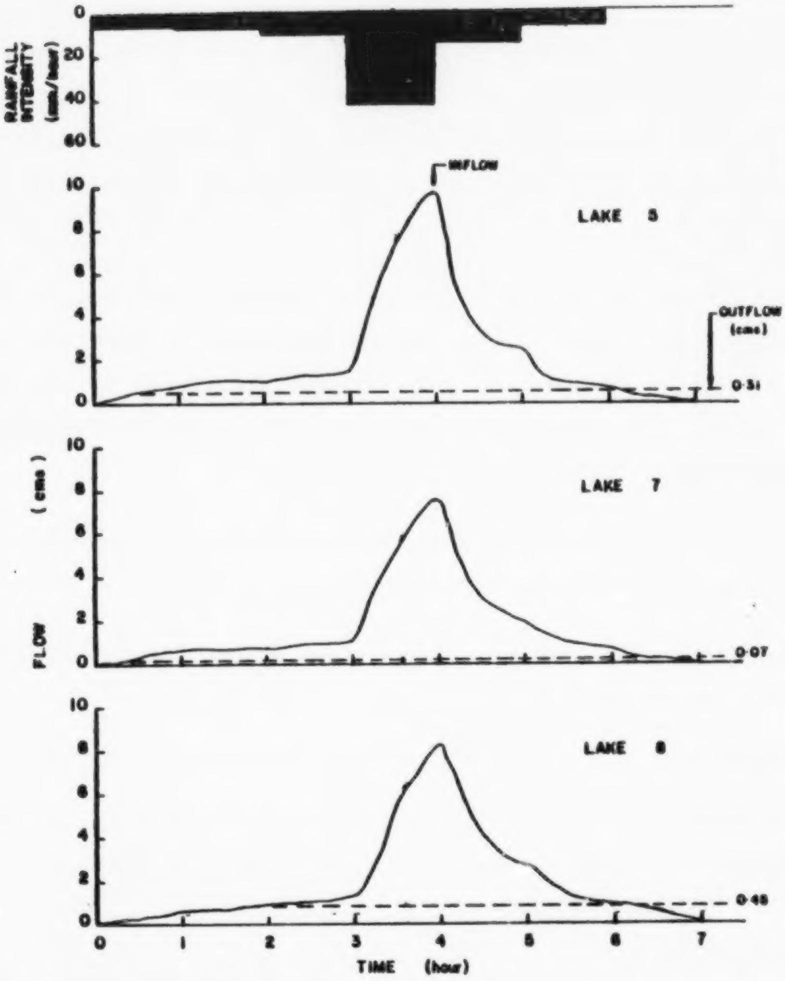

FIG. 6a HYDROGRAPH FOR 100-YEAR 6-HOUR STORM

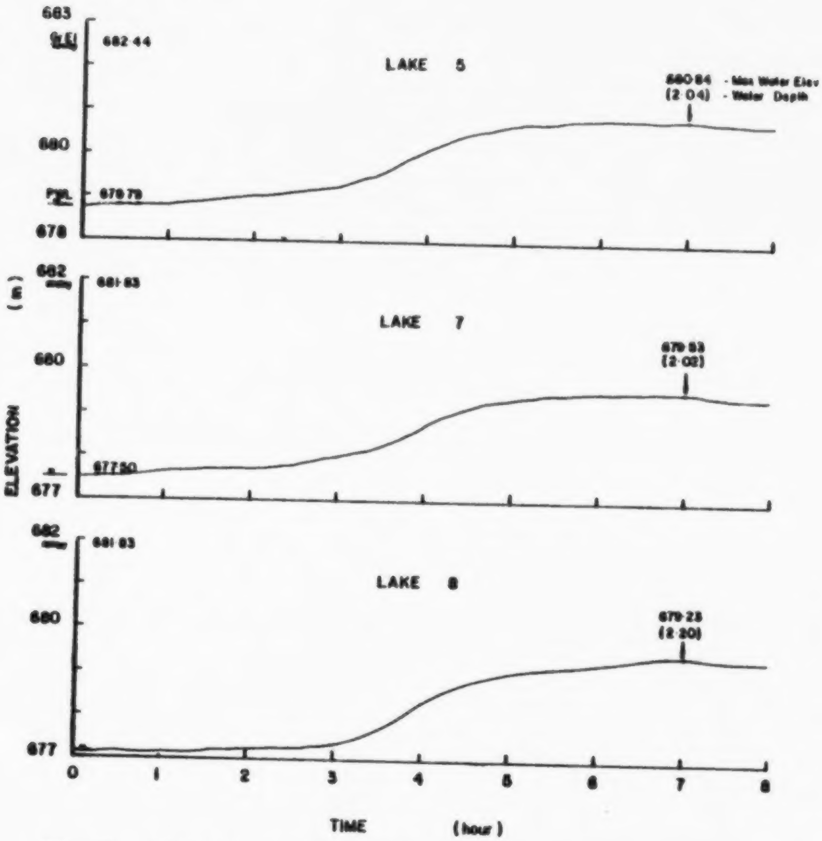

FIG. 6b LAKE LEVEL VARIATION FOR 100-YEAR 6-HOUR STORM

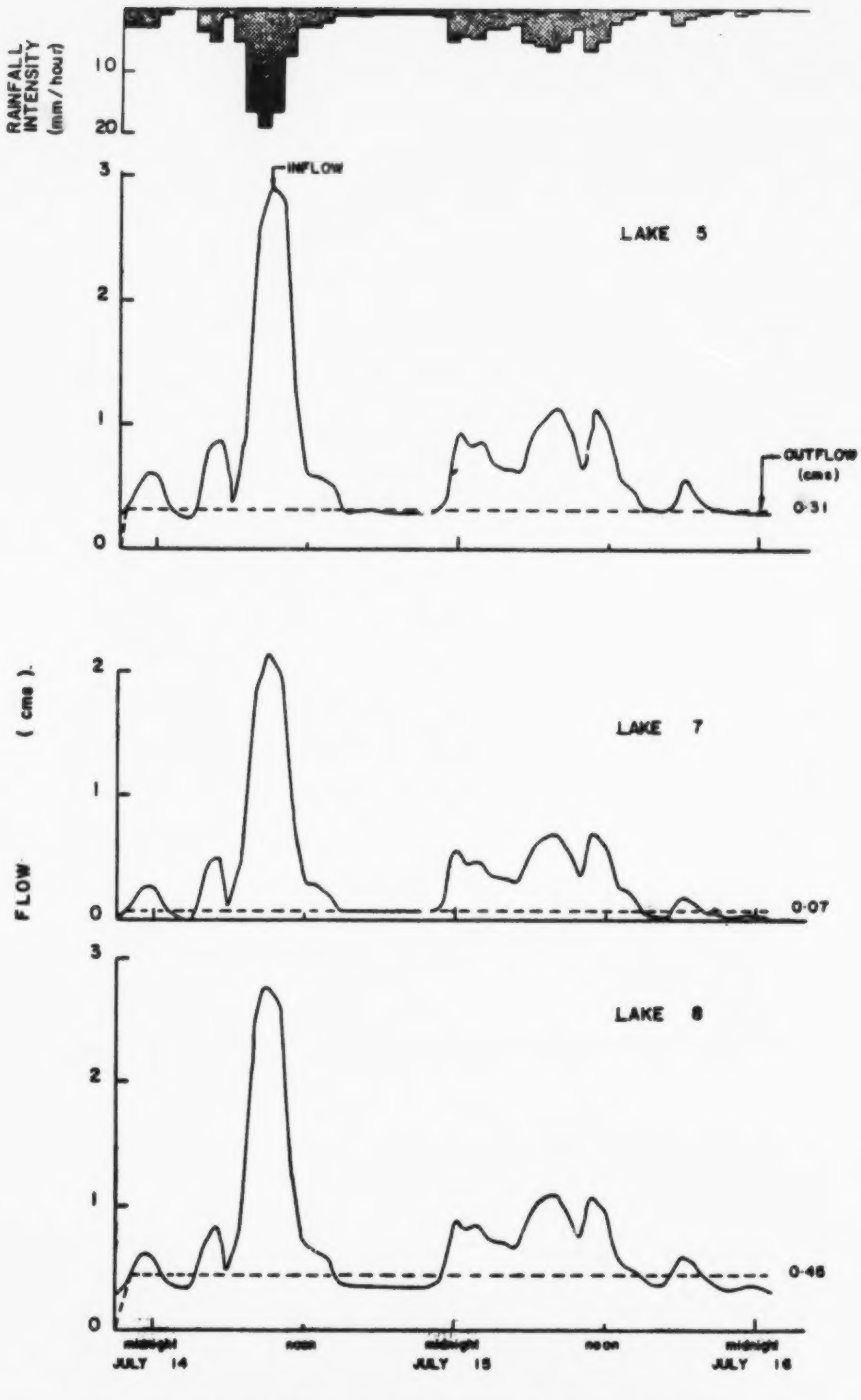

FIG. 7a HYDROGRAPH FOR JULY 14-15, 1937 STORM

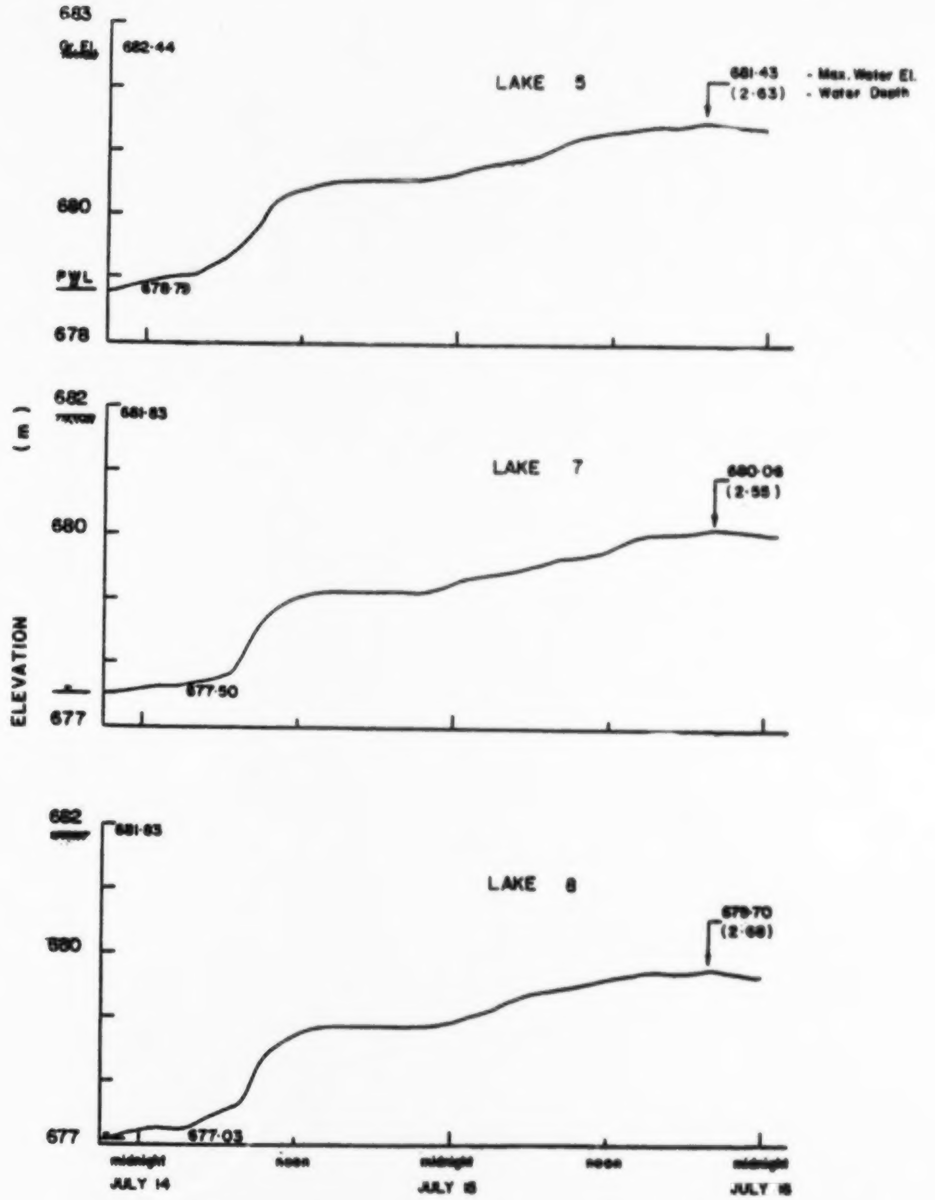

FIG. 7b LAKE LEVEL VARIATION FOR JULY 14-15 STORM

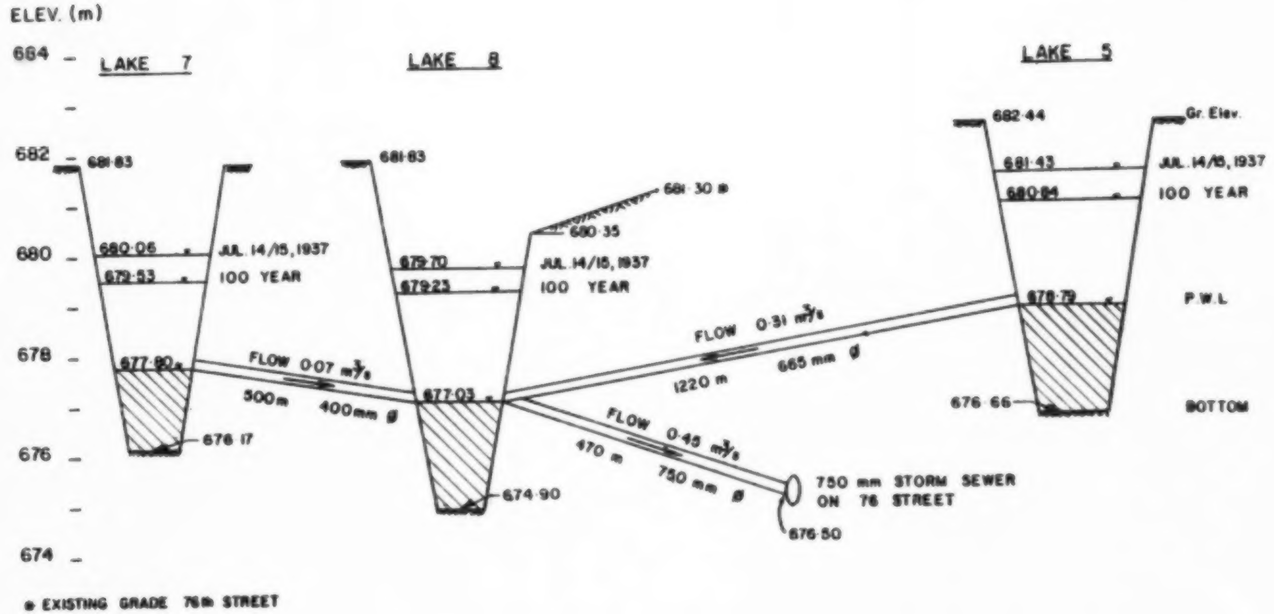
ELEV. (m)
684
682
680
678
676
674
LAKE 7
LAKE 8
LAKE 5
681·83
680·06
679·53
677·80
676·17
JUL. 14/15, 1937
100 YEAR
681·83
679·70
679·23
677·03
674·90
681·30
680·35
682·44
681·43
680·84
678·79
676·66
Gr. Elev.
P.W.L
BOTTOM
FLOW 0·07 m³/s
500m 400mm ø
FLOW 0·31 m³/s
1220 m 665 mm ø
FLOW 0·45 m³/s
470 m 750 mm ø
750 mm STORM SEWER ON 76 STREET
676·50
* EXISTING GRADE 76th STREET

FIG. 8 THREE LAKE SYSTEM HYDRAULIC PROFILE

The SWMM Sub-Model Runoff was used in the optimization design. This model computes overland flow hydrographs for a given storm pattern for each subcatchment in the drainage area. It allows for the variation in infiltration before and during runoff occurrence.

The study area was subdivided into numerous tributary subcatchments ranging from 0.5 to 7 ha in size. The characteristics of each subcatchment, such as overland flow length and slope were directly measured and input into the runoff model. The imperviousness values for various residential land uses were determined from the size and density of housing conditions as indicated in Table 3. The distribution of various imperviousness conditions on the area can be found in Figure 2. The overall a erage percentage of imperviousness in the whole drainage area was estimated to be 42%.

4.2 Comparison of SWMM and HYMO

Since the two hydrological models had been used at various stages in this and previous studies, a comparison between SWMM and HYMO was undertaken to determine the best model for refining the analysis in this study. The results of the comparison are presented in Table 4 for several design storm events.

TABLE 3

COMPARISON OF COMPUTED RUNOFF FROM SWMM AND HYMO MODELS

STORM			SWMM*			HYMO**		
Frequency (year)	Duration (hour)	Rain Volume (mm)	Peak Flow (m^3/s)	Runoff Volume (cms-day)	Runoff Vol. / Rain Vol. (%)	Peak Flow (m^3/s)	Total Volume (cms-day)	Runoff Vol. / Rain Vol. (%)
5	2	32.2	10.3	0.46	40.0	6.0	0.28	24.0
5	6	44.5	7.6	0.60	38.5	7.4	0.54	34.3
100	6	90.2	25.5	1.70	53.3	23.5	1.80	56.3
July 14-15, 1937:								
48 hr.		157.5	7.3	2.60	45.0	10.7	4.20	70.3
max. 6 hr.		65.3	7.3	1.07	47.0	10.7	1.56	63.4
max. 24 hr.		103.6	7.3	1.57	43.6	10.7	2.33	63.0
July 15, 1937 only		55.9	2.5	1.03	52.0	4.9	1.97	99.0

* Imperviousness used in SWMM is ± 42%
** Complex number used in HYMO is ±85

TABLE 4

BASIC HYDRAULICS WITH VARIOUS STORM EVENTS

Storm		Runoff Volume to Lake				Max. Lake Storage Used				Max. Water Depth Above Lake PWL			Max. Water Surface Area at Lake			
Frequency (Year)	Duration (Hours)	5	7 (cms/day)	8	Total	5	7 (cms/day)	8	Total	5	7 (m)	8	5	7 (ha)	8	Total
A. Infrequent Storms																
100	6	.656	.495	.554	1.706	.636	.479	.538	1.653	2.05	2.03	2.20	2.93	2.52	2.59	8.04
July 14-15, 1937		.991	.771	.846	2.608	.836	.640	.689	2.165	2.64	2.56	2.68	3.23	2.75	2.81	8.79
B. Frequent Storms																
1	2	.082	.062	.069	.213	.072	.056	.062	.190	0.30	0.29	0.38	2.15	1.72	1.77	5.64
2	2	.098	.079	.085	.262	.089	.072	.079	.239	0.38	0.35	0.47	2.17	1.76	1.82	5.75
5	2	.174	.134	.151	.459	.164	.128	.144	.436	0.68	0.66	0.80	2.33	1.88	1.96	6.17
10	2	.249	.194	.213	.656	.239	.187	.207	.633	0.90	0.88	1.03	2.44	1.98	2.07	6.49
25	2	.453	.351	.387	1.191	.443	.344	.380	1.168	1.55	1.51	1.70	2.73	2.28	2.36	7.37
5	6	.231	.197	.179	.607	.213	.165	.182	.560	.81	.80	.93	2.38	1.96	2.03	6.37

The analysis showed that both SWMM and HYMO provided very close results for the 100 year and 5 year 6 hour duration storms which were adopted for lake sizing.

However, the SWMM runoff volumes range from 38.5% to 53.3% of the rainfall volume for various storm events while the HYMO volumes vary from 24 to 99%. It appears that SWMM provides a more consistent runoff volume considering the 42% imperviousness of the drainage area.

On July 5, 1937, (2nd day of the 48 hour storm), HYMO generated a runoff volume of almost 100% of the rainfall. This means that no infiltration was considered by HYMO on this day. The rainfall record (upper part of Figure 3B) shows that there was no significant rainfall for almost 9 hours after the rainfall of the first day and hence recovery of soil infiltration should be considered. This finding indicates that HYMO is not very suitable for long duration flow simulation purposes.

As a result of these results, it was concluded that the SWMM Runoff Model was more suitable for the present system analysis. This conclusion appears to be consistent with the HYMO Model's origin as a rural, single event simulator.

However, it cannot be recommended, at this stage to replace HYMO by SWMM runoff for all other urban drainage analyses. Our experience indicates that after revising several parameters, (such as TP, K, CN), HYMO can also provide satisfactory results.

4.3 Hydraulics

- Geometry of Lakes

  The geometry of the proposed detention lakes have been designed according to a number of engineering and planning factors. These include the shape of the existing sloughs and depressions, space limitations due to street and housing planning, and the location of the inlet and outlet of storm sewers.

  Kidney shaped lakes (Figure 9) are proposed with side slopes of 7:1

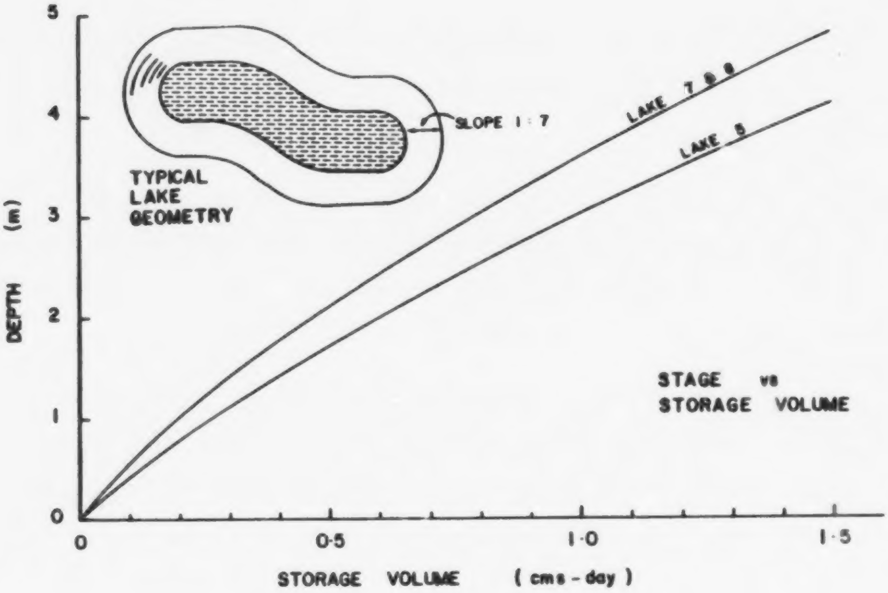

FIG. 9 STORAGE CHARACTERISTICS OF TYPICAL LAKES

and surface areas of 1.6 and 2.0 HA. at permanent water levels (P.W.L.). The storage characteristics of typical lakes are plotted on Figure 9. The shape of lakes may be altered in the final design provided there is no significant change in their hydraulic characteristics as shown on Figure 9.

- Hydraulic Computer Model

The SWMM sub-program, extended transport (EXTRAN), was used to analyze the design and operation of the detention lake system. This model was used to route hydrographs through the drainage system (including lakes) to the point of discharge. The routing technique used is very sophisticated and it gives reliable results. The lakes were modelled as very large trapezoidal open channels connected by small closed conduits.

The trapezoidal "Lake" elements in the model were 60 m wide with side slopes of 8.5:1. Within the hydraulic conditions under investigation, (1.6 and 2.0 HA. surface areas at P.W.L. and less than 3 m water depth variation) no significant variation in the hydraulic characteristics of the lakes was found. During the hydraulic simulation, no instability was observed and the accuracy is believed to be within 5%.

A special modelling technique was applied for simulating a hydro brake concept at the most downstream lake (Lake 8). The flow at the outlet of the lake system was controlled to a constant rate of 0.45 cms during all storm events simulated.

REFERENCES

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2. A. Fok et al, "Application of Stormwater Management Concepts for a Master Drainage Planning Study", a paper submitted to Sixth International Symposium on Urban Storm Runoff, July 1979, Kentucky, U.S.A.

ALTERNATIVE URBAN FLOOD RELIEF MEASURES

A CASE STUDY

CITY OF REGINA, SASKATCHEWAN, CANADA

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1 - INTRODUCTION

The state of the art of computer simulation in hydrology, specifically in urban drainage, has advanced significantly in recent years. This has been a result of the increase in the useage of such models in studies and design, along with the advancement in the sophistication and the efficiency of the models.

However in many cases these advanced tools and methodologies are being used to analyze and design conventional or traditional solutions to urban drainage problems. Probably the most common solution to solve combined sewer overflows and flooding is either large scale sewer separation in older urban areas, or the installation of larger conduits in areas where a separate system presently exists.

A recent report by the Comptroller General of the United States (December 28, 1979) stated that "large construction projects to correct combined sewer overflows are too costly". The report stated that estimates by the Environmental Protection Agency to curb pollution caused by sewer overflows are $26 billion, and $62 billion to prevent flooding. It noted that progress in stemming pollution and flooding has been slow, with neither Federal Government nor local communities capable of supplying the funds for these large construction projects. The report goes on further to recommend that new low-cost techniques should be investigated and encouraged prior to considering costly solutions.

2 - CASE STUDY

A study was recently completed by our firm in which a series of alternatives to correct flooding problems in an existing urban area were developed. These ranged from conventional solutions, the installation of larger piping, to alternative storm water management measures.

2.1 Background

The City of Regina, Saskatchewan Canada, has been plagued over the years by frequent flooding of basements and subways within the Seventh Avenue drainage area. Such events have caused inconvenience to both home owners and vehicular travel, in addition to the cost associated with the flooding damage. Other resulting problems which cannot be as readily assessed are the health dangers when sanitary sewage backs up into basements, as well as the personal grief experienced by homeowners and the environmental damages to the receiving stream during combined sewer overflows at the pumping station. Furthermore, if any re-development is to proceed in the downtown core, which is located partially within the study area, improvements to the existing system will be required.

The City of Regina, recognizing these existing problems and the need to correct them, commissioned a study in the spring of 1978 to evaluate the present design criteria utilized for storm sewer design. In summary, the findings of the study concluded that:

(1) Present storm sewers have a one year storm frequency capacity.

(2) Further engineering studies would be required to determine the cause of, and the most effective remedies for the inadequate storm drainage system.

As a result of the latter conclusion, the City of Regina requested proposals and our firm was engaged to carry out the required study.

2.1.1 Terms of Reference

The City of Regina did not provide detailed terms of reference for the study, but to allow maximum freedom of approach established the following guidelines:

(1) To investigate the cause and the magnitude of the deficiencies in the existing storm drainage system.

(2) To provide an economical upgrading plan for the system.

(3) To provide preliminary cost estimates to implement the plan.

2.1.2 Objectives

The objectives of this study were established after review of the terms of reference and discussions with the City of Regina's Departments of Public Works and Engineering. They were as follows:

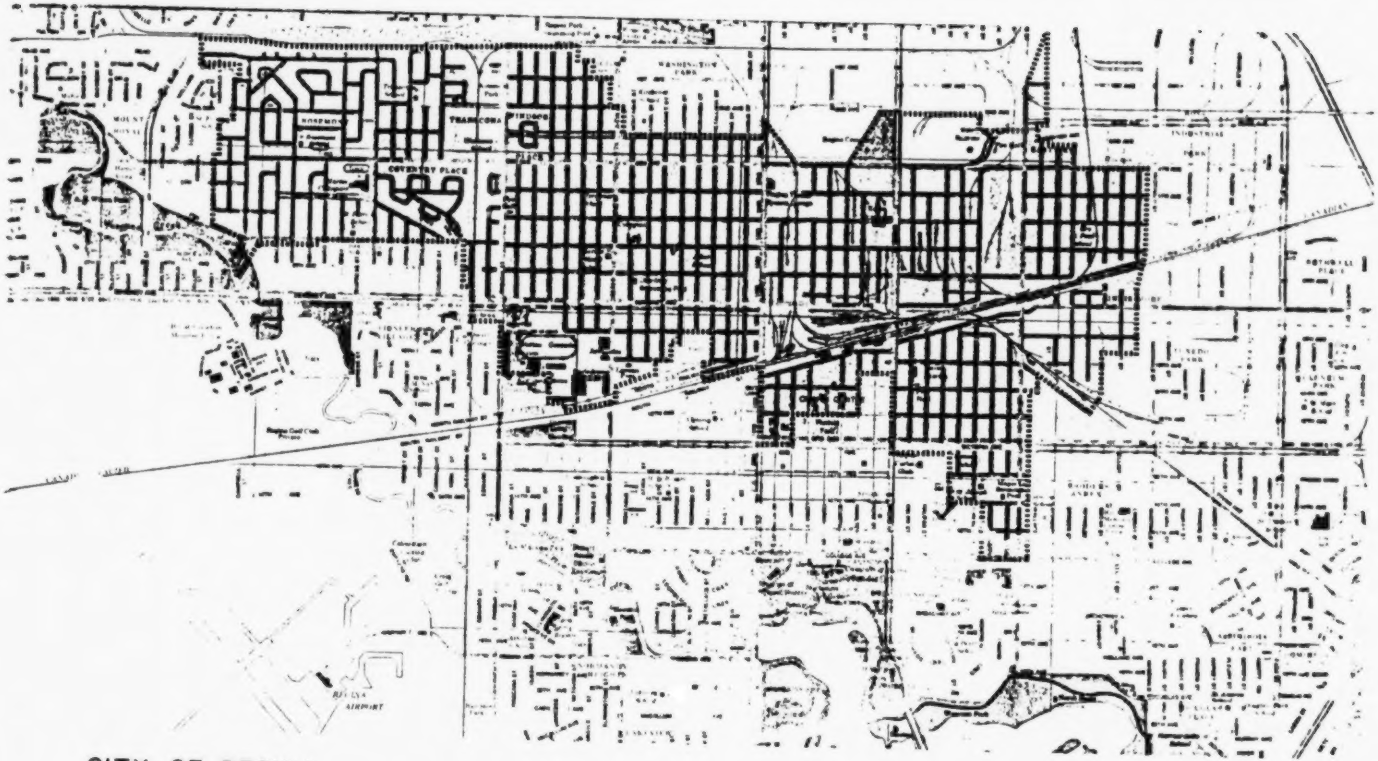
(1) To determine the cause and the magnitude of the deficiencies of the existing storm and sanitary sewer network serving the Seventh Avenue drainage area.

(2) Provide an economical upgrading plan for the system, achieving at least a five year protection against basement flooding and reducing the frequency of street and subway floodings.

(3) Provide an upgrading plan for the system that recognizes and accounts for continued development in the downtown area.

2.2 Description of Study Area

The study area known as the Seventh Avenue drainage area encompasses approximately 1,040 hectares, as shown in Figure 1. Drainage for the site is through a separate sewer system, namely storm and sanitary sewers. The trunk sewers are primarily located along Seventh Avenue and drain in a westerly direction towards the Wascana Creek. The storm trunk outlets directly to the Wascana Creek, while the sanitary trunk sewer is pumped below the creek and is lifted to the sewage treatment plan. Sewers along minor roadways outlet to the trunk sewers, and generally have a perpendicular orientation to the trunks. Close examination of the sewer system in the course of the study indicated that the system is not truly separate as the name implies. Numerous interconnections between the storm and sanitary sewers occur throughout the study area. Common manholes exist throughout the drainage area, which result in direct storm water inflow during storm conditions to the sanitary sewer, and ultimately in basement floodings. Further, approximately 50% to 67% of the roof areas within the residential and industrial areas respectively are discharged directly to a sewer which, in the majority of the study area, is the sanitary sewer, thereby directing additional storm runoff to the sanitary



CITY OF REGINA

SEVENTH AVENUE DRAINAGE AREA

FIGURE 1.

▪▪▪▪▪▪▪▪▪ DRAINAGE BOUNDARY

system. Also the reported frequency of basement and surface flooding indicates that the sewer network is often overloaded, resulting in surcharged conditions.

2.3 Computer Simulation

The hydraulic complexities of the existing sewer system dictated that a sophisticated urban hydrologic-hydraulic computer simulation program be utilized in this study. Traditional design methods are incapable of dealing with such complex networks, nor can they be utilized to evaluate advanced storm water management upgrading schemes. Thus the Hydrograph Volume Method (HVM) program was selected for the analysis.

The basic modelling procedure was to segmentize the trunk sewer system in detail, manhole to manhole, including the common manhole interconnections. The relatively simple tree-type configuration of the local sewers consisting of long straight sections permitted a more coarse segmentation, while still maintaining the hydraulic characteristics of the sewers.

Initially the existing sewer system was evaluated through computer simulation in order to assess its hydraulic performance under actual and design storm events. The results indicated that the system is heavily overloaded for storms of a 2-year return frequency with severe overloading for the 5-year return frequency, with the conveyance capacity of the system being at a 1-year level or less.

2.4 Evaluation of the Existing System

Considerable storm water overflows were indicated to be occurring to the sanitary trunk at common manholes. In addition, roof water contributions were quickly overloading the existing sanitary sewer along local streets. The present conveyance capacity of the sanitary trunk is only 2.5 times the design dry weather flow, or alternatively the peak dry weather flow, with three trunk segments requiring some minor upgrading.

Similarly, the storm sewer system is incapable of conveying the storm runoff flows, resulting in roadway flooding, and overflows to the sanitary sewer. In portions of the study area where weeping tiles are discharged to the storm sewers, surcharging in the storm sewer would lead to basement heaving, cracking and flooding. Also a restriction in the storm water flow occurs at the outlet to the Wascana Creek, both by the limited capacity of the sewer and turbulence at the junction chamber

2.5 Minor Remedial Works

Having determined that storm water overflows at common manholes and the restricted outlet to the Wascana Creek are prime contributors to the flooding problems, the effectiveness of incorporating remedial work to rectify these problems were evaluated. This was achieved through the computer simulation of these alternatives.

The results indicated that a significant reduction in the trunk sanitary flows will result, and backwater levels in the storm sewer immediately upstream from the Wascana Creek outlet would be reduced. These remedial works, however, will not eliminate basement floodings for the 2 and 5-year storms, but will reduce the severity of the flooding. The effectiveness of these works will increase in significance for the lesser storms. Also the wet weather flows reaching the treatment plant will be reduced, thereby lessening the probability of overflows to the Wascana Creek and the volume of waste water reaching the treatment plant.

Since these measures were not sufficient to upgrade the sewer network to an adequate level of service,additional alternatives were evaluated.

2.6 Generation of Alternatives

A series of individual alternative upgrading schemes were generated for the storm and sanitary system. These were under three general groupings:

(1) Conventional upgrading; consisting of the replacement or addition of piping to convey the peak flows for either the 2-year or 5-year return frequency storms.

(2) Total storage; the provision of storage within the sewer system, such that flows are limited to the existing sewer's capacity. The storage would be provided in off-line and in-line subsurface tanks.

(3) Combination system; conventional upgrading in the trunk sewers, with storage to prevent flooding on streets served by local sewers.

All the alternatives included the elimination of common manholes and the increased storm outlet capacity to the Wascana Creek.

2.6.1 Roof Downspout Disconnection Modifications

Roof downspouts connected to either the storm or sanitary sewers contribute significantly to the peak flows conveyed by these sewers. Although volumetrically their total contributary area may be relative minor to the total storm runoff, their instantaneous high peaks may cause floodings, or structural damages. The effectiveness of downspout disconnection was evaluated, through the resimulation of the above alternatives with all downspouts assumed to be discharged to the surface.

2.7 Evaluation of Alternatives

Through the course of the study, nine individual alternatives were developed, six being for the storm system and three for the sanitary system. From this series of individual alternatives, twenty-seven combination upgrading schemes were developed as shown in Tables 1 and 2.

The findings of the study indicated that conventional solutions, namely those requiring the replacement and the addition of sewers, were the most costly and would result in greatest inconvenience and disturbance during the construction phase. Approximately 45% and 61% of the existing sewer system would require upgrading for the 2 and 5-year storms respectively. Further, the resulting peak flows outletting to both the sewage pumping station and the Wascana Creek would be greater than those presently experienced. This will result in the degradation of the Wascana Creek, through erosion and overflows at the pumping station.

Alternatives incorporating storage proved to be the most cost-effective. The total storage alternative requires no major upgrading to the existing sewer network, other than some minor piping to convey flows to the storage facilities. These storage facilities for the most part would be subsurface storage tanks. However, where surface ponding such as roof tops, parking lots, or open spaces is feasible, they will be utilized to reduce subsurface storage requirements. In addition to being the least expensive of the alternatives, other benefits derived are a reduction of peak flow to the Wascana Creek and the sewage pumping station, thereby possibly improving conditions in the Wascana Creek.

The combination system did not result in any significant benefits, with the high costs associated with upgrading of the trunks negating any benefits derived from the storage.

TABLE 1

ALTERNATIVE UPGRADING SCHEMES

STORM SYSTEM

Conventional Upgrading
5 Year $41.1 M (1)
2 Year $21.2 M (2)

Conventional Upgrading
(all downspouts to storm sewers)
5 Year $41.9 M (3)
2 Year $21.8 M (4)

Conventional Upgrading
(trunk sewers only)
Storage for Laterals
5 Year $28.2 M (5)
2 Year $15.0 M (6)

Conventional Upgrading
(trunk sewers only)
Storage for Laterals
(all downspouts to storm sewers)
5 Year $30.0 M (7)
2 Year $16.7 M (8)

Total Storage
5 Year $19.8 M (9)
2 Year $6.2 M (10)

Total Storage
(all downspouts to storm sewers)
5 Year $22.5 M (11)
2 Year $9.0 M (12)

SANITARY SYSTEM

Conventional Upgrading
5 Year $14.4 M (a)
2 Year $12.2 M (b)

Downspout Disconnection
5 Year $0.5 M (c)
2 Year $0.5 M (d)

Total Storage
5 Year $8.1 M (e)
2 Year $5.0 M (f)

All costs in millions of dollars

Downspout disconnection is by far the most cost-effective solution for the upgrading of the existing sanitary sewer. Elimination of this source of storm water inflow will limit flows in the sanitary sewer to the peak dry weather flow, which the system has sufficient capacity to convey.

The recommended solution called for the elimination of all common manholes and other interconnections, with storm outlet improvements to the Wascana Creek. Upgrading measures would require that all downspouts be disconnected with storage being provided within the storm sewer system.

TABLE 2

COST ESTIMATES FOR VARIOUS UPGRADING SCHEMES

5 Year Storm + 5 Year Sanitary		2 Year Storm + 2 Year Sanitary		2 Year Storm + 5 Year Sanitary	
1a	$55.5M	2b	$33.4M	2a	$35.6M
1e	$49.2M	2f	$26.2M	2e	$29.3M
3c	$42.4M	4d	$22.3M	4c	$22.3M
5a	$42.6M	6b	$27.2M	6a	$29.4M
5e	$36.3M	6f	$20.0M	6e	$23.1M
7c	$35.0M	8d	$17.2M	8c	$17.2M
9a	$34.2M	10b	$18.4M	10a	$20.6M
9c	$27.9M	10f	$11.2M	10e	$14.3M
11c	$23.0M	12d	$ 9.5M	12c	$ 9.5M

3 - CONCLUSION

The case study presented here clearly indicates that solutions to combined sewer overflows and basement floodings do not necessarily require large costly construction projects. Rather, with some innovativeness, low cost techniques can provide a much more cost-effective solution.

A LONG-TERM DATA BASE FOR THE INVESTIGATION OF URBAN RUNOFF POLLUTION

BY

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INTRODUCTION

Realistic conclusions concerning receiving water pollution can not be derived from rainfall-runoff measurements or simulations of singular events. This is especially true of intermittent storm runoff and the resulting storm or combined sewer overflows [1] . The numerous highly interdependent variables which influence the runoff process, and the random nature of storm events necessitate the investigation of long-term records. Further, the rapid variation of precipitation and runoff necessitates close observation of the runoff process. This requires continuous and detailed monitoring of the rainfall-runoff-overflow process over a time period of several years. Precipitation, runoff and overflow measurements are usually reported in literature for individual storm events.

This paper describes an instrumentation and monitoring program collecting rainfall, runoff, outfall and overflow data continuously in a sequence of closely spaced intervals in two test catchments: one a combined sewer system of 1340 acres (542 ha), the other a small separate system of 57 acres (23 ha). Particular emphasis is placed on the difficulties in establishing and analyzing such an extensive data base. The project, funded by the German Research Society, commenced in 1974 [2] . Continuous monitoring started in 1976 and is ongoing.

STUDY OBJECTIVES

The prime objective of this study is to establish a continuous and detailed data base of rainfall and runoff, both quantity and quality. On such a basis the following tasks are planned:

- determination of seasonal and annual totals for urban runoff quantity and quality and respective combined sewer overflow and separate system outfall figures;
- definition of antecedent moisture conditions and pollution potential in the study areas;
- some indication of the influence of areal distribution of rainfall on the runoff process;
- identification of the different parameters influencing the runoff process and definition of the basic principles of the quality mechanisms existent in the rainfall-runoff process;
- description of the runoff process as mathematical algorithms in a deterministic, parametric, and stochastic way;
- verification of different deterministic, parametric and stochastic mathematical approaches.

The objectives of this study tie neatly into the findings and recommendations of last year's design storm seminar, held in conjunction with the SWMM-Users' Group meeting in Montreal, which suggested that

- for development of alternatives to the design storm concept a continuous data base is required;
- further research should concentrate on antecedent moisture conditions; and
- for consideration of qualitative aspects the amount of pollutants present at the beginning of a storm event is of prime importance [3].

HARLACHING TEST CATCHMENT

Selection Criteria

The criteria for selection of the major test catchment for the study were the following:

- the drainage area had to be large enough to be representative of an urban combined sewer system;
- for valid correlation between rainfall and runoff multiple diversions or a large number of intercepting points were not desirable;

- the catchment slopes had to be relatively flat and the site accessible in order that monitoring equipment could be installed;
- traffic and public disruption had to be kept to a minimum;
- the site should be a combined sewer overflow in order to allow for direct derivation of overflow figures from the data; and
- the site had to be sufficently large to allow for installation and maintenance of equipment.

The Harlaching drainage area met these criteria and was selected for sampling.

Catchment Characteristics

The Harlaching catchment is part of the Munich drainage area and has a combined sewer system (Figure 1). Located at the southern edge of the city, it covers an area of 1340 acres (542 ha). 39 % of the area is impervious, of which 15 % is roofs and 24 % streets and sidewalks. Land surface slopes vary up to 1.7 %. The land-use is mainly residential with a population density ranging from 12 to 80 inhabitants per acre (Figure 2). The area includes some small commercial sections, two hospitals, playgrounds, and schools (Figure 3).

The catchment boundaries are clearly defined with the exception of a few interconnections to the adjacent drainage system located at the northwestern catchment boundary (Figure 4). Most of these interconnections were completely closed off for the study; four had to be left partially open in order to prevent local flooding under severe storm conditions. The southern catchment boundary is formed by a forest and the western boundary is formed by a recess in the land surface adjacent to the Isar River. Changes in the nature of the area as well as in the number of inhabitants during the course of the investigation are negligible.

Rainfall Monitoring

There are three rainfall recording stations within the catchment. Their locations are indicated on

Figure 1:

Munich drainage area indicating the monitored catchments of Harlaching and Pullach

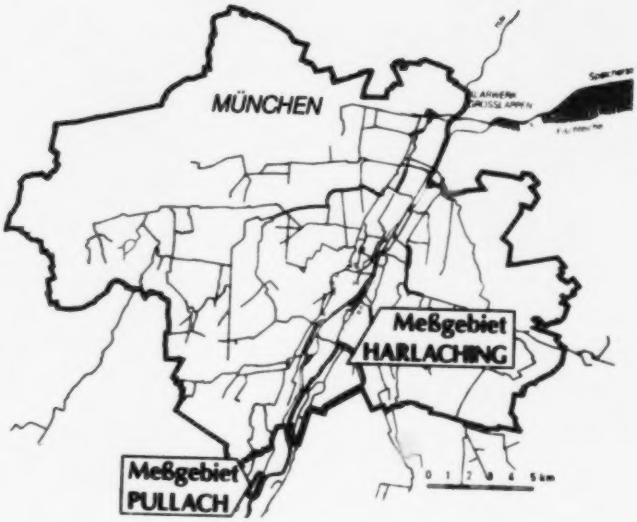

MÜNCHEN
Meßgebiet
HARLACHING
Meßgebiet
PULLACH
0 1 2 3 4 5 km

Figure 2:

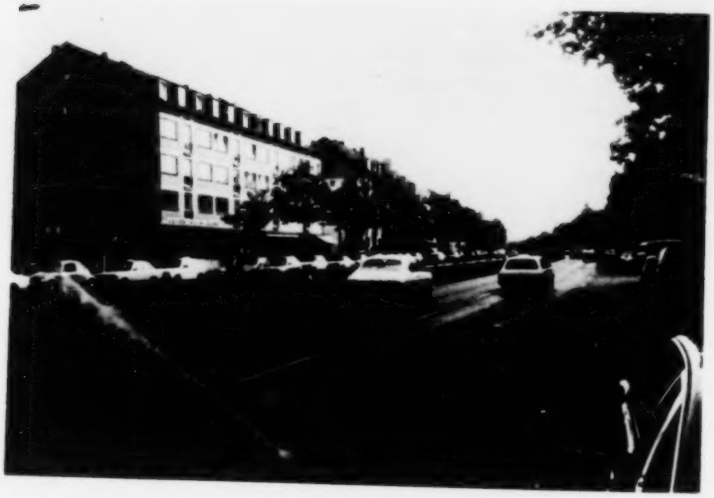
Harlaching residential area

Figure 4 as triangles. The individual stations are equippend as follows:

- Harlachinger Str./Wettersteinstr.	1 Hohenpeißenberg type Ombrometer 1 Hellmann device 1 Totalizer
- Am Hollerbusch	1 Hohenpeißenberg type Ombrometer
- Harlaching hospital	1 Hellmann device

The three types of rainfall recorders just mentioned may briefly be described as follows:

Figure 3:
Harlaching commercial area



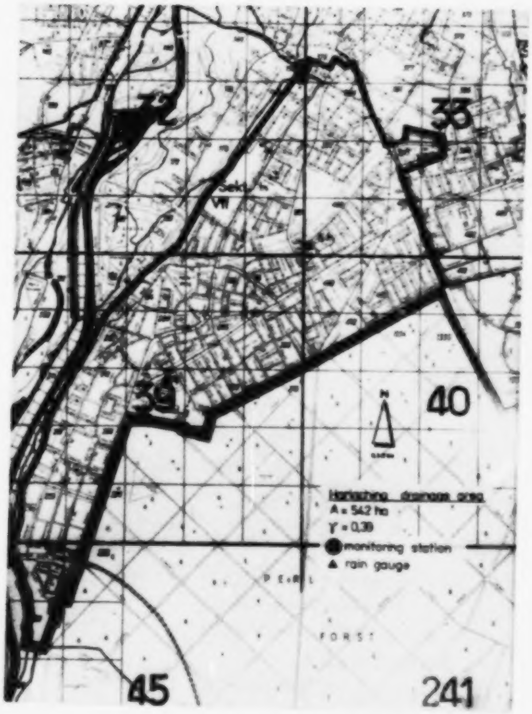

33
39
40
Harlaching drainage area
A = 542 ha
γ = 0.39
● monitoring station
▲ rain gauge
PERL
FORST
45
241

Figure 4:
Harlaching catchment

- The Hellmann device records cumulative rainfall depth as indicated by a float, mounted in a container, which is situated underneath a catching funnel. The recording cylinder advances at a rate of 6 in/hr (16 mm/hr).
- Components of the Ombrometer type are a standardized receiving container, a drop dispenser that transforms precipitation into drops of uniform size, and a light barrier, which produces a signal according to the number of drops. Usually the gauge is equipped with a clock and a digital printer, which gives results as time, and number of drops per minute. The rainfall intensity is calculated from a calibration curve.
- The Totalizer rain gauge stores precipitation and furnishes precipitation totals.

Under continuous operation, all rainfall recorders used functioned faultlessly. The reliability of the precipitation measurements, however, was generally influenced by the cleanliness of the catching funnels. For the Ombrometer it should be noted that under very high rainfall intensities a continuous stream is formed rather than individual drops. This offsets the light barrier drop counting principle, and this data is therefore unusable for the very high rainfall intensity range.

For 1976 the rainwater caught in the totalizer has been analyzed for total suspended solids, COD, total phosphorus, Ammonia-nitrogen (NH_3), Nitrite-nitrogen (NO_2), Nitrate-nitrogen (NO_3), conductivity, and pH.

Runoff Monitoring

All runoff monitoring and sampling was done at the overflow site, the area's intercepting point, shown on Figure 4 as a solid circle. The time of concentration to this point is approximately 40 minutes at a maximum sewer length of approximately 3 miles (4500 m), and an average sewer slope of 0.5 %. The combined sewer overflow structure is situated underneath a parking lot (Figure 5). Therefore it was possible to establish the sampling station on top of the manhole leading down to the structure. The shed holds a field laboratory and above-ground instrumentation.

Figure 5:

Shed of Harlaching monitoring station on top of combined sewer overflow structure

Figure 6 portrays the overflow structure looking downstream prior to the installation of a working platform. The runoff peak of a storm event with a once per year frequency amounts to about 350 cfs (10 m^3/s). Overflow starts at 20 cfs (o.6 m^3/s). Figure 7 shows the working platform looking upstream toward the inflow of the structure.

For runoff, in addition to flow depth, the following quality parameters are continuously recorded in 5-minute-intervals: temperature, conductivity and turbidity. The flow is determined by water levels measured at three different points:

- at the entrance to the overflow;
- at the exit to the conduit leading to the treatment plant; and
- in the overflow pipe leading to the Isar River.

The water depths are measured by ultrasonic echo sounders (Figure 8). A frequency of 50 kHz generates a very narrow beam, which provides for clear measurements, even at a very low water level. In addition, the devices operate without touch-contact. The depth-flow relationship was established by tracer measurements. The water level recorders operate with a minimum number of failures. The instrument indicators have to be checked and verified annually.

Wastewater turbidity provides an indication of the pollution level in general. For turbidity different angles of dispersion are measured: 0^o, 25^o, 90^o and 135^o. The equipment itself functions faultlessly. Up until now, however,

Figure 6:

Harlaching overflow structure prior to the installation of monitoring equipment

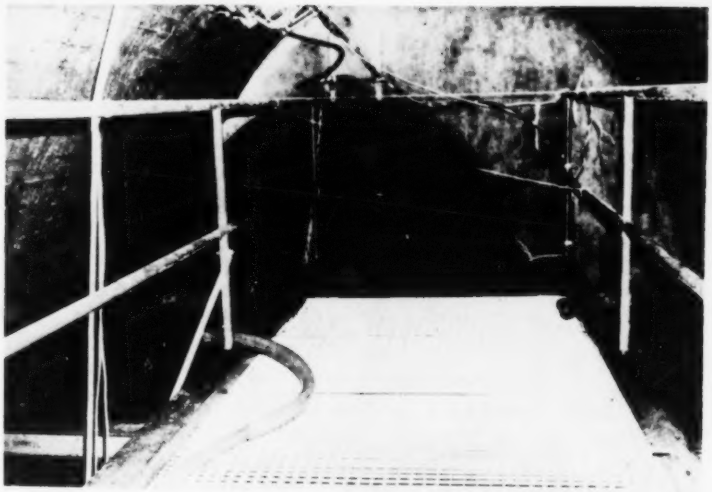


Figure 7:

Harlaching overflow structure after installation of a working platform

there have been frequent periods where the turbidity equipment was inoperative due to blockage in the connecting pipes.

The conductivity of the wastewater is a measure of the content of dissolved, dissociated inorganics and organics. The conductivity changes significantly during runoff and is proportional to the content of dissociated inorganic salts. Knowledge of the salt content and variations thereof in storm runoff is important for both the operation of treatment plants and for the living conditions of microorganisms in receiving waters. Conductivity is measured with a palladium electrode. The device functions free of maintenance exept that it is important to clean the sensor twice a week.

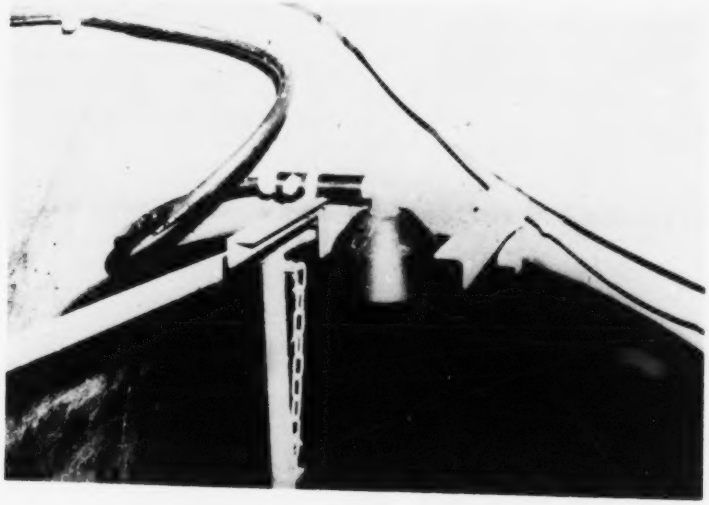


Figure 8:

Ultrasonic echo sounder for water depth measurement at the entrance to the Harlaching overflow structure

The temperature is measured over a thermosensor. With regular maintenance inoperative periods for this sensor have thus far been avoided.

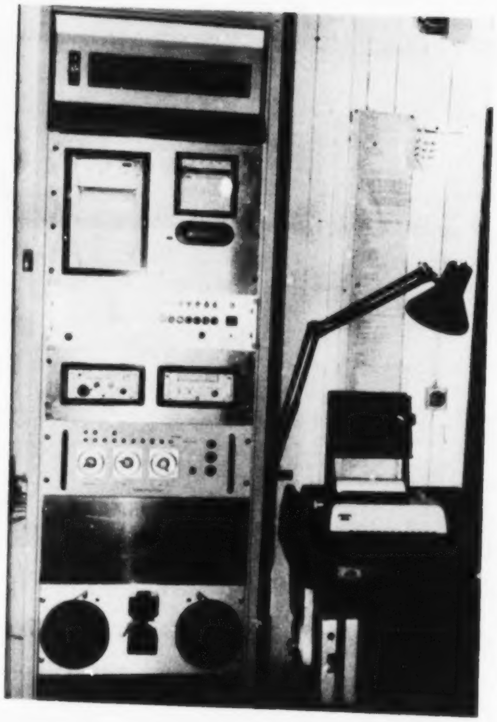
A small computer was installed in the station in order to allow for (Figure 9):
- continuous monitoring;
- on-line data recording to enable the immediate control and review of the gathered data; and
- providing the data in a form ready for processing to minimize work and to avoid mistakes.

The central processing unit (CPU) of this computer has a core of 32 k bytes. Half of the core is occupied by the operating system, which uses the process language BASEX, an extension of BASIC. A teletype featuring a keyboard, tape punch and reader is attached through an interface. Besides monitoring and coordinating the on-line data which were the three flow depths, turbidity, conductivity and temperature, this computer has to:
- check the data for plausiblity and provide warnings in case of malfunctions;
- record the rainfall data collected next to the monitoring station with a scanning frequency of one minute; and
- survey and operate the sampler and a pumping system related to the sampler.

The sampler itself is of special design There is a continuous sample flow which has the advantage that blockages in the sampler conduit can be recognized in time and removed without the loss of samples. For this a flush-back set-up was

Figure 9:

Computer and teletype at the Harlaching monitoring station



installed which washes out the conduit with tap water by reversing the pump direction. The flushing is triggered automatically by the computer. The sampler has a capacity of 160 one liter bottles, one liter is approximately 1 quart, which are accomodated in 20 metal baskets, each with 8 bottles (Figure 10). The sampler operates by chain advancement. Conseryation of the samples is achieved through cooling to 4°C. The positioning of the bottles under the filling mechanism occurs automatically (Figure 11). The position of the bottles in the sampler is recorded through binary coded mechanical contacts by the computer and is coordinated to the filling-time. The filling of the bottles is achieved through free fall, using a flow diverter in the sample stream. At each sampling time two bottles are filled. The time and duration of this action is controlled by the processor. Sampler overflow is collected in an open channel and diverted back into the sewer. A circuit-breaker is built into the drain channel in order to prevent flooding in case of line blockage.

Under dry weather conditions the time lapse between two samples amounts to 90 minutes; under storm conditions it

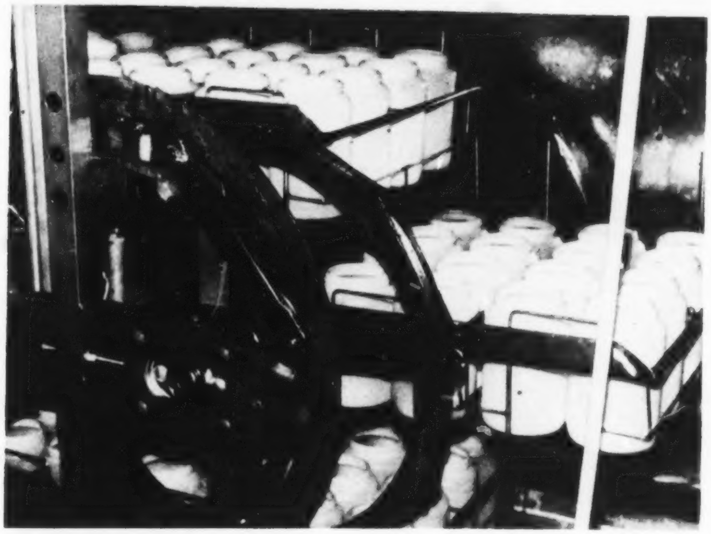


Figure 10

Bottle baskets of sampler at the Harlaching monitoring station

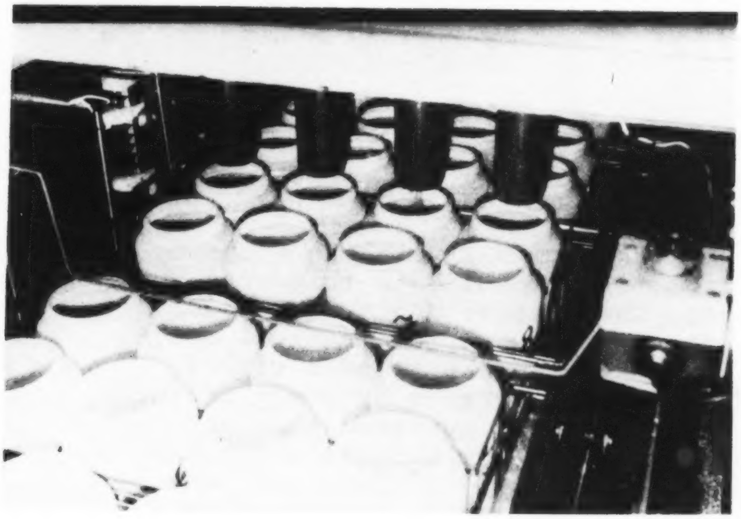


Figure 11:

Filling mechanism of sampler at the Harlaching monitoring station

is 5 to 20 minutes. Storm runoff conditions are recognized and double checked by the processor using the rainfall measurements next to the station and the water level in the conduit leading to the treatment plant.

The sampling interval during a storm event is controlled by the rate of change of the "turbidity load". The turbidity load is defined as the product of turbidity measured at the 0° angle and flow.

The laboratory analysis program encompasses six parameters: total suspended solids, COD, and Kjeldahl-nitrogen on all samples, and BOD, total phosphorus and total organic carbon on every fifth sample. This program has been in effect since 1977.

For relating runoff to rainfall and quality to quantity, the exact timing of each sample is of considerable importance. The sampler, the continuous pollutant recordings, the water level measurements, and the one rainfall recorder are all exactly synchronized with the aid of the computer. Exact timing in the other rainfall recorders is kept with a quartz watch.

For an overview Figure 12 portrays the complete data collection system of the Harlaching monitoring station [4] . Shown is the continuous sample flow drawn from the sewer passing trough the turbidity measurement of 0° angle and through the sampler. From the continuous sample flow the flows for the turbidity measurements of 25°, 90° and 135° angles are branched off. A data flow chart is incorporated into this figure as dashed lines: the different signals from the recorders - three for water depth, four for turbidity, one each for temperature, conductivity and rainfall - feed into the computer. The slah-dotted lines indicate the path of active control information, as for example to control pump and sampler operation.

PULLACH TEST CATCHMENT

Selection criteria

As mentioned above, measurements are also taken in a smaller separate system with similar land-use and topographical characteristics. This was done to obtain information on storm runoff without dry weather flow in order to aid analyzing and separating the individual components of combined runoff in the other catchment. The Pullach separate drainage system was selected.

Catchment Characteristics

The Pullach drainage area is located adjacent to Munich's southern border and lies on the high bank of the Isar River. This catchment is 57 acres (23 ha) in

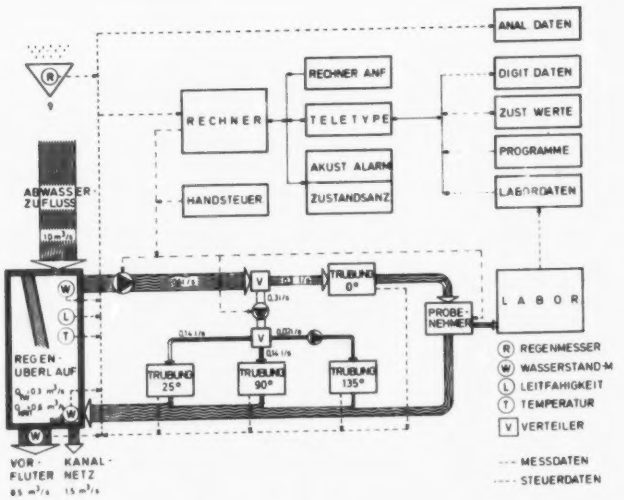

ANAL DATEN
RECHNER ANF
RECHNER
TELETYPE
DIGIT DATEN
ZUST WERTE
PROGRAMME
AKUST ALARM
ZUSTANDSANZ
HANDSTEUER
LABORDATEN
ABWASSER ZUFLUSS
10 m³/s
0,31/s
0,14 l/s
0,021/s
0,14 l/s
TRUBUNG 0°
TRUBUNG 25°
TRUBUNG 90°
TRUBUNG 135°
PROBE-NEHMER
LABOR
REGEN-UBERL AUF
Q 0,3 m³/s
Q 0,6 m³/s
VOR-FLUTER 8,5 m³/s
KANAL-NETZ 1,5 m³/s
R REGENMESSER
W WASSERSTAND-M
L LEITFAHIGKEIT
T TEMPERATUR
V VERTEILER
MESSDATEN
STEUERDATEN

Figure 12:

Data collection system of the Harlaching monitoring station

ABWASSERZUFLUSS	wastewater inflow
AKUST. ALARM	acoustic signal
ANAL. DATEN	analog data
DIGIT. DATEN	digitized data
HANDSTEUER.	manual control
KANALNETZ	sewer system
LABOR	laboratory
LABORDATEN	laboratory data
LEITFÄHIGKEIT	conductivity
MESSDATEN	measured data flow
PROBENEHMER	sampler
PROGRAMME	programs
RECHNER	computer
RECHNER ANF.	computer requirements
REGENMESSER	rainfall recorder
REGENÜBERLAUF	combined sewer overflow
STEUERDATEN	active control information
TELETYPE	teletype
TEMPERATUR	temperature
TRÜBUNG	turbidity
VERTEILER	divider
VORFLUTER	receiving waters
WASSERSTAND-M.	flow depth recorder
ZUSTANDSANZ.	status indicator
ZUST. WERTE	status figures

size (Figure 1). The land-use is residential and includes a small central commercial section, a playground and a school. 36 % of the area is impervious, of which 18 % is roofs and backyards, and 18 % streets and sidewalks. Land slopes are similar to those in Harlaching and the concentration time ranges from 22 to 12 minutes for rainfall intensities ranging between .14 and 1.4 in/hr (10 and 100 l/s ha). A storm sewer outfall to the Isar River provides the sole intercepting point in the area. The sampling station is located at this point (Figure 13). All catchment boundaries are clearly defined. The groundwater table is approximately 130 ft (40 m) below the surface, ensuring that groundwater does not infiltrate into the sewer lines. The drainage system has not undergone any changes significant to the investigation.

Rainfall Monitoring

In the Pullach drainage area there is one rainfall recording station with all three types of recorders mentioned before: a Hellmann device, an Ombrometer, and a Totalizer. The tiny circle on Figure 13 indicates the location of the rainfall station within the catchment.

As in the Harlaching catchment the rainwater caught in the totalizer in 1976 was analyzed for total suspended solids, COD, total phosphorus, Ammonia -nitrogen (NH_3), Nitrite-nitrogen (NO_2), Nitrate-nitrogen (NO_3), conductivity and pH.

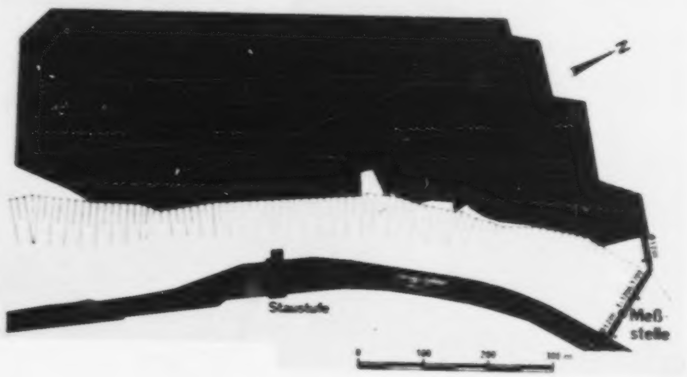


Figure 13:
Pullach catchment

- o rain gauge
- ● storm runoff monitoring station

Runoff Monitoring

The monitoring station of the Pullach storm sewer system is situated in the stilling section of the main stormwater collector 250 ft (75 m) behind the drop into the Isar Valley.The construction of the measuring station, with installations for direct measurements of flow depth, conductivity, temperature and pH and for collecting stormwater samples, is portrayed in Figure 14.

Flow is measured by determining the water depth at a Venturi, which was built into the channel. An influence on the data due to downstream backwater in the channel can not occur, as the outflow of the storm sewer is above the high-water level of the Isar River. An air bubble method is utilized in order to determine the water depth. The conversion of water depths into flow is done according to a calibration curve. The continuous flow record is plotted by advancing the velocity of the paper 2.4 in/hr (60 mm/hr). This yields a resolution of one minute. This measuring procedure functions quite reliably.

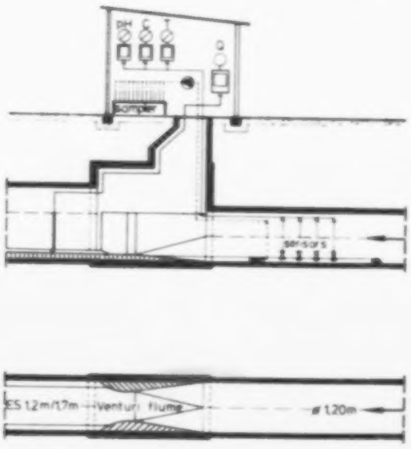


Figure 14:

Schematic view of the Pullach monitoring station

C	conductivity
ES	egg shape
pH	pH-value
Q	flow
T	temperature

In order to measure conductivity and temperature, equipment similar to that employed at the Harlaching monitoring station was used.

Stormwater samples are collected downstream of the Venturi flume, insuring complete mixing of the flow. Constant back-up in the flow section is forced by a sill obstruction, 5.5 in (14 cm) high in order to guarantee sampling at the starting and final proportions of the storm runoff process when flow depths are low, and in order to prevent air entry into the pumpline. The pump was placed on a working platform where it forced the sampling flow into a sample collection device above ground surface. The sampler has a capacity of 22 bottles of 1.5 quarts (approx. 1.5 l) each.

In 1977 and 1978 there have been two programs for sampling: one was to draw mixed samples, that is filling only one or a limited small number of sample bottles per event. Here the sample was automatically compounded from different samples drawn at variable spacings, the timing of which varied from 5 to 60 minutes and depended on the flow rate.

The other program was to fill individual sample bottles, which occurred automatically at time intervals of 5 to 60 minutes, depending on the flow rate.

The frequent sampling in both catchments resulted during rainy seasons in a quantity of samples which exceeded the capacity of laboratory. As one of the major intents of the data collection in the separate system of Pullach was to define antecedent moisture and pollutant conditions, and as these figures also may be derived from the continuous flow recording and mixed samples, the Pullach monitoring program was changed to only draw mixed samples - that is where only one or a few sample bottles are filled per rainfall event.

The samples were analyzed for settleable solids, total suspended solids, BOD, COD, Ammonia-nitrogen (NH_3), Nitrite-nitrogen (NO_2), Nitrate-nitrogen (NO_3) and total phosporus.

DATA RECORD

Existing Data

The amount of data collected per year is considerable. In 1977 and 1978 approximately one million pieces of data were collected, and therefore data storage and retrieval had to be completely computerized.

The data acquired by the small local computer of the Harlaching monitoring station are punched on paper tape and then fed into a data bank established at the University computer center. All laboratory data are listed and punched for data bank entry.

Data checks for format and plausibility are employed prior to finally establishing the data sets. The reliability of data can be impaired by various factors which include external influences, and malfunctioning or maladjustment of the measuring equipment. The data correction proved to be very difficult and is for the most part attainable only through human intervention. In this case, analog recordings are of the greatest value.

The best overview of the existence of data is provided by plots showing the number of measurements available per day or week. Figure 15 and 16, for instance, are an illustrative example from 1979, when the monitoring station was out of service for six weeks due to an act of arson. Figure 15 shows the existence of directly processed data, figure 16 the existence of laboratory data.

The goal of a truly continuous data base can never be met. Data gaps of different lengths occurred due to malfunctioning of individual recorders and, more seriously, due to malfunctions of the small processor at the measuring station or of the sampler. Major sources of disruption of the monitoring program, aside from the above mentioned act of arson, have been pump failures and motor defects in the sampler. Statistical analysis of the data gaps showed that rainfall data are available at an almost continuous basis, while runoff data include gaps of up to 35 days in 1977, up to 15 days in 1978 and up to 56 days in 1979. Those figures refer to the data of the combined sewer system of Harlaching. In total it is estimated that about 50 % of

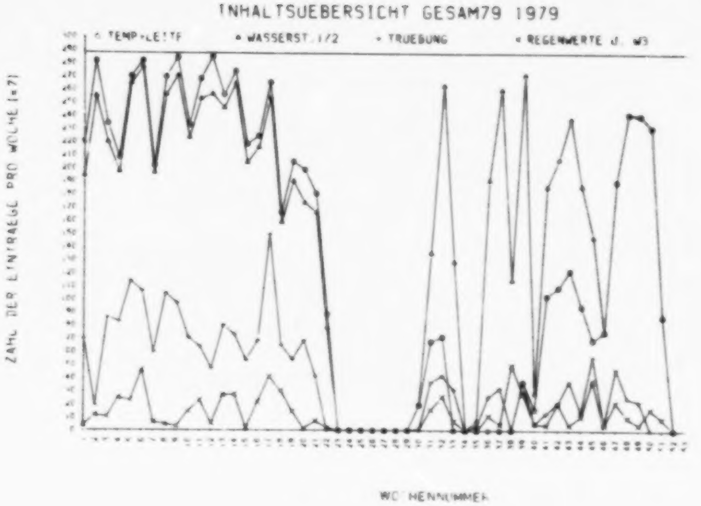


Figure 15:

Existence of directly processed data

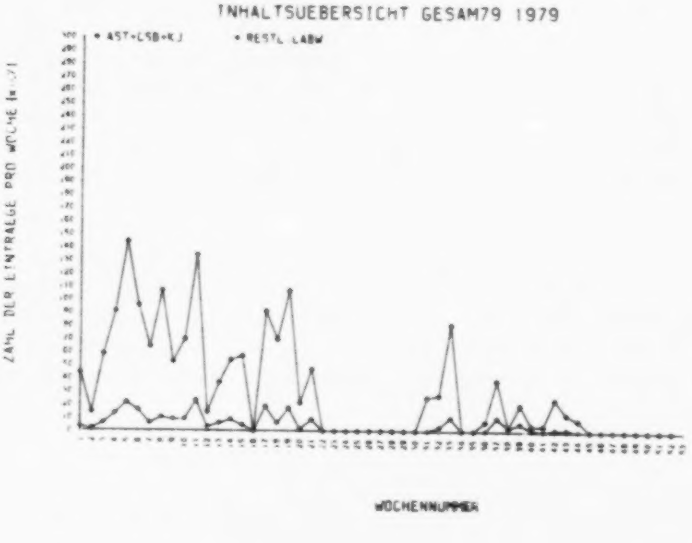


Figure 16:

Existence of laboratory data

AST + CSB + KJ	TSS and COD and Kjeldahl-nitrogen
INHALTSUEBERSICHT GESAM 79	content 1979
REGENWERTE U. W3	rainfall data and flow depth 3
RESTL. LABW	other laboratory data
TEMP + LEITF	temperature and conductivity
TRUEBUNG	turbidity
WASSERST. 1/2	flow depth 1 and 2
WOCHENNUMMER	number of week
ZAHL DER EINTRAEGE PRO WOCHE	number of figures per week

the runoff data are recorded, referring to 100 % being a truly continuous data set.

Data supplement

Still, a complete data base is required for the purpose of:

- defining annual and seasonal pollutant loads contained in runoff and overflow;

It is also desired for

- statistical analysis of rainfall-runoff-overflow data with the aim of defining durations and frequencies; and for
- performing a stochastic analysis of the rainfall-runoff-overflow process.

Therefore, the gaps must be filled, an effort which is being undertaken at present. The seriousness of a gap is judged upon by comparison of the gap's duration with the intended recording spacing. Minor gaps are interpolated from the existing data. It is intended to employ rainfall-runoff simulations, using the complete rainfall data as input, to produce runoff figures, both for quantity and quality, to fill the major gaps. Such substitution may be trusted, as there are sufficient data available to calibrate rainfall-runoff models for the test catchments.

At the beginning it was mentioned for the Harlaching catchment that for high intensity storms there might be more than the one interception point recorded. This uncertainty will also be checked by employing rainfall-runoff simulations. Data in question then may be replaced by simulation results.

LEGAL AND REGULATORY ASPECTS

The study is scheduled to be completed by the end of 1982 and is quite important in Germany for legal and regulatory reasons:

- communities are required by law to furnish detailed master plans in order to obtain allowances to discharge stormwater into receiving waters [5] ;
- a law enacted in 1976 requires communities to pay fees for wastewater loading [6] . This includes stormwater discharged into receiving waters.

This "Waste Water Fee Law" is of primary importance. It aims for a higher level of water quality and to more justly distribute the cost burden for the prevention of, removal of, and compensation for water pollution. The mandatory fee does not take effect until December last of 1980. After this date the fee will be levied for so-called pollution damage units. With respect to stormwater discharges and combined sewer overflows this law asserts that:
- the number of pollutional units for storm water only which, via public drainage, is discharged into receiving waters is equal to 12 % of the total population figure - for instance, for a city like Munich that is a fee of 3.6 million $ US/year. The treatment plant effluent is taxed separately.
- the individual states determine to what degree this number of pollutional units shall be diminished, if storm water runoff or combined sewer overflows are retained or treated.

In Northrhein-Westphalia, for example, this specific regulation reads as follows [7] :
- The number of pollutional damage units for storm water discharges shall be reduced by 50 % if storm runoff from a separate system is retained and treated for a time period of at least 20 minutes. For combined sewer overflows they shall be reduced if the system is designed so that overflow does not start until the rainfall rate exceeds .07 in/hr (5 l/s ha).
- Direct storm runoff is free of charge if it is treated in a settling tank for a time period of at least 20 minutes. For combined sewer overflows there is no charge if the system is designed so that overflow does not start until the rainfall rate exceeds 2 in/hour (15 l/s ha).

The basic principle on which the procedures are based to dimension overflows and retention basins properly is,according to another regulation, the concept of catching the "first-flush" related pollution [8] . The data on which this concept is based, were derived from a limited number of measurements taken in a small combined sewered catchment [9] .

Hence, the entire body of laws, regulations and guidelines lacks a well-defined data base. For German conditions this emphasizes the necessity of a continuous dense data base as established in this study.

SUMMARY AND CONCLUSIONS

The necessity for a continuous and simultaneously dense urban runoff data base is shown for technical and regulatory reasons. Therefore rainfall and runoff quantity and quality were monitored in two catchments, 57 acres (23 ha) and 1340 acres (542 ha) in size, since 1976. For quality mainly total suspended solids, BOD, COD, Kjeldahl-nitrogen, total phosphorus and total organic carbon were analyzed.

The study, sponsored by the German Research Society, showed the limits of automation in urban runoff monitoring. Frequent checking of the monitoring equipment and manual comparison of directly computerized data with orginal recordings proved to be necessary.

The goal of a truly continuous field data base hardly can be met. Data gaps of different lengths occured mainly due to malfunction of equipment. It is suggested to fill gaps in the collected runoff data with simulation results to provide a complete data base for statistical analysis and derivation of annual and seasonal runoff-overflow figures.

It is recommended to establish further long-term monitoring programs to advance the knowledge in urban hydrology.

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